ECONOMİC ANALYSİS AND TYPE SELECTİON FOR
A HİGHWAY BRİDGE

A Thesis

Submitted in Partial Fulfillment of the requi-
rements For the Degree of Master of Science in Civil Eng.

By

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OUTLINE OF MY THESIS FOR MY M.S. AT GEORGIA SCHOOL OF TECHNOLOGY

1. TITLE — Economic Analysis and Type of Selection Between a Concrete and a Steel Highway Arch Bridge.

2. PREFACE

3. INTRODUCTION

4. DESIGN OF THE CONCRETE ARCH BRIDGE

    a. SLAB

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    b. FLOOR SYSTEM

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8. DRAWINGS

9. ECONOMIC COMPARISON

10. CONCLUSION AND SPECIFICATIONS

approved

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PREFACE

The purpose of this thesis is to give a complete design for a three hinged steel arch and a reinforced concrete arch highway bridge; both for the same crossing.

To make an economic comparison of these bridges and select the bridge which will fit the site the best.

It is wished to express sincere appreciation to the following men: Prof. F. C. Snow and Prof. J. N. Smith of the Georgia School of Technology for their kind assistance given throughout the preparation of the thesis.

All the information for the design of the two arches were taken from the following books: " Movable and Long Span Bridges, (Vol. III)" , Reinforced Concrete Construction, Hool (Vol. III)" ,"Hanbook of Building Construction, Hool and Johnson, (Vol. I)" ; " Roofs and Bridges, Part IV Merriman and Jacoby" ; " Notes on the Design of Concrete Structures, Prof. Snow of Georgia School of Technology" , " Economics of Highway Bridge Types, Mc. Cullough."

Most of the design computations are given as slide-rule results, if there are some small errors, the results are beleived to be accurate enough for all practical purposes.

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INTRODUCTION

The bridges referred to in this thesis have been designed for a proposed crossing over the Sakarya river near Bolu, small town of Turkey. The location of this crossing is not of importance, but a bridge is neccessary to avoid a great detour leading to the main highway from Istanbul to Ankara, capital of Turkey. So in designing these bridges the question of aesthetics was almost not considered. The main purpose of this thesis was the design of the concrete arch and the steel arch and an approximate cost of the two bridges was neccessary in selecting the most economical one for the same crossing.

The designs are only complete enough to determine the most economical structure, but detailed designs have not been attempted

The concrete arch and the steel arch have first been treated separately and later a comparison has been made between the two bridges.

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PART I

DESIGN OF THE REINFORCED-CONCRETE ARCH BRIDGE

SPAN = 140 FT.

RISE = 20 FT.

DESIGN OF CONCRETE ARCH BRIDGES

A concrete arch as ordinarily constructed, that is, with fixed ends, is statically indeterminate, but it can be analyzed by taking into account the elasticity of the material. The arching is nothing more than a curved beam and is considered accordingly.

It would seem from purely theoretical considerations, that but little could be gained by the use of reinforcement in a concrete arch, since the direct compression usually controls to such an extent that the allowable stress in the concrete permits of but a small unit tensile stress in steel. From a broader point of view, however, it is clear that the steel adds greatly to the reliability of the construction and makes possible a higher working stress in the concrete than could properly be employed in the design of plain concrete structures. Higher working stress produces a thinner arch ring and consequently less dead loads and lighter abutments. Undoubtedly, a large saving may result from this cause in case of long span arches.

A considerable portion of an arch ring is subject to both positive and negative moments, and for this reason the reinforcement should be placed, for some distance at least, near both upper and lower surfaces. The general practice is to carry both rows of steel throughout the entire span, thereby eliminating any possibility of failure due to an inadequate provision for tensile stresses. An account of the heavy compressive stress in an arch design of the rings, the upper and lower reinforcements should be tied together to prevent the buckling. Before the elastic theory can be app--

lied to arch design, either the dimensions of the arch ring must be assumed outright or the thickness at the crown made to conform to some empirical formula. With the arch thus approximately designed the stresses may then be computed computed by the elastic theory.

There are different methods of arch analysis based on the elastic theory. In the book " Concrete Structures " written by Urquhart and O'Rourke the origin of the coordinates of the arch, is taken at the crown. The method of the " Bureau of Public Roads " places the origin of the coordinates of the rings at the springing line.

The arch treated in this thesis has been designed according to the method of the " Bureau of Public Roads. "

The arch to be designed has a span of 140 ft. and a rise of 20 ft. which makes the ratio of span over rise equal to 7.

The thickness at the crown is found by using the M. J. W. Douglas formula in the book " Reinforced Concrete Construction, Page 20 ".

$h = .0001\ (\ 11{,}000 + l^2\ )$ Check this formula

where

h = the thickness of the crown

l = the span

The thickness of the springing line has been taken as 2.25 times the thickness of the crown. 2.25 x 3.06 = 6.85 ft.

DESIGN OF THE FLOOR SYSTEM

Highway bridge slabs must be designed to carry the heavy concentrations brought upon them by the wheels of Modern Motor Trucks. The question arises at once as to the width of slab which supports any given concentrated load. Fig. A illustrates the case, where a wheel is shown resting at the center of a wide slab, supported along the edges ab and cd. The strip of slab, of width T, beneath this wheel cannot deflect under the load without causing at the same time a deflection of the adjecent strips, and in this way the effect of the load is distributed over an indefinite width. several experimenters have studied the problem, and on the basis of their data and results various rules have been proposed to be used in designs.

The following slab design has been based on Ketchum's formula namely $W = 2/3S + T$, and the following data has been used for the whole design: H = 15 loading, Impact 30% of live load,

Concrete 2000lb, (1,2,3), $n = 15$, $6\frac{1}{2}$ slump, maximum aggregate 1 inch.

$f_c = 800$ lb/in^2

$f_s = 20{,}000$ lb/in^2

$u = 100$ lb/in^2

Curbs 9in x 12in above surface of paving

Paving 2in Bituminous, Wearing Surface.

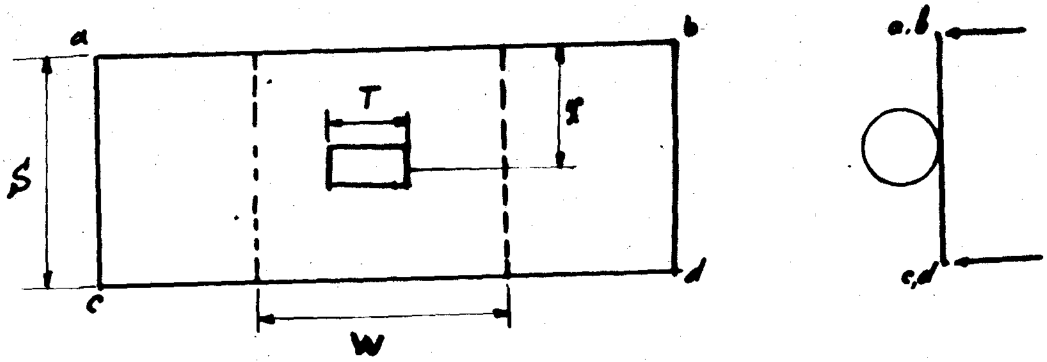


fig. A

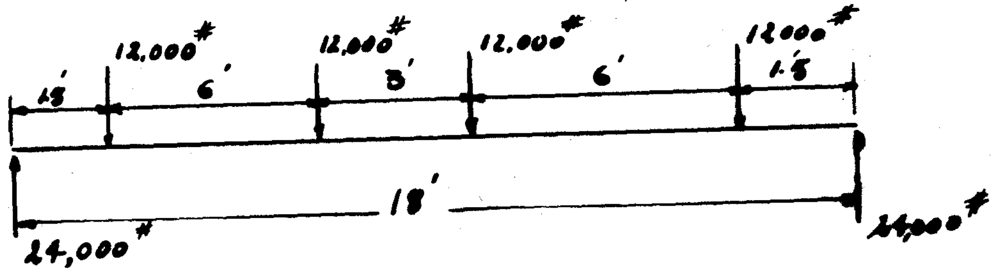


fig. C

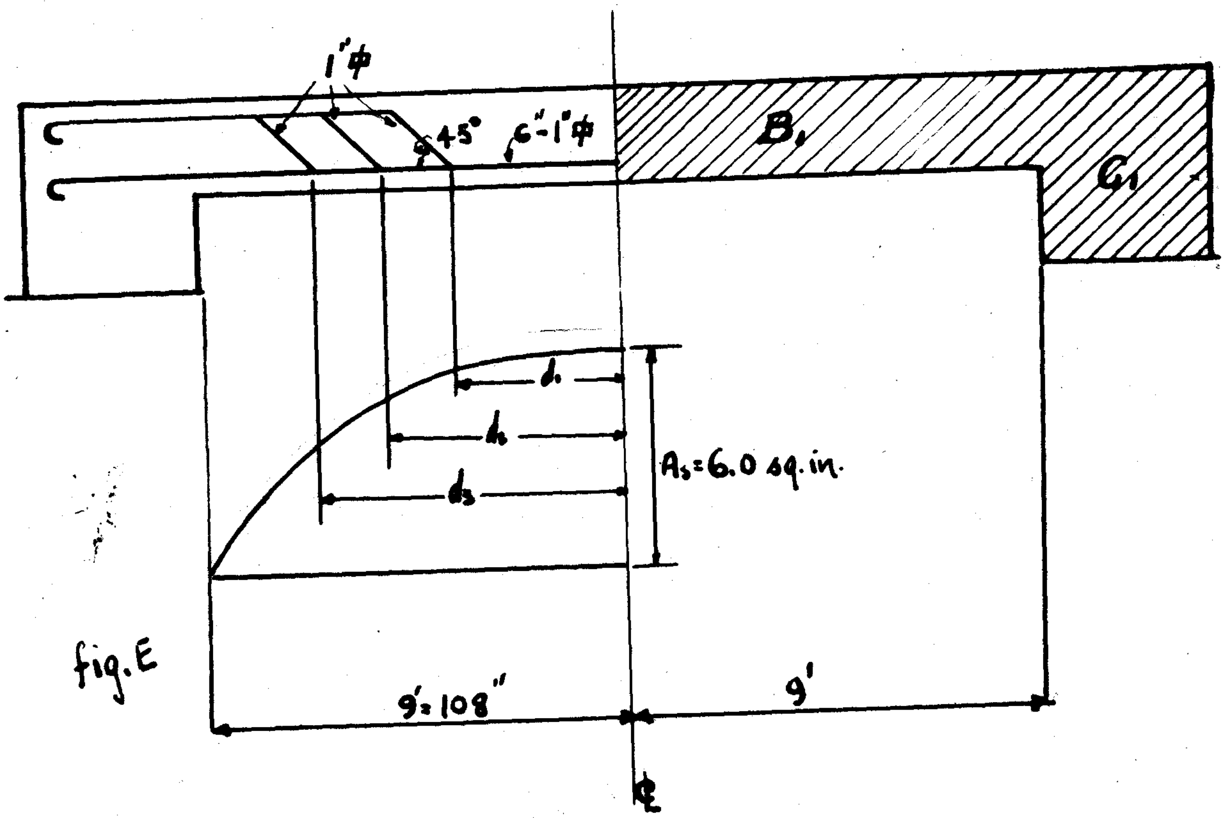


fig. E

SLAB DESIGN

The use of Ketchum's specification was chosen to determine the width of the slab which supports one wheel. The moment and the shear were computed for a strip of slab 12 in. wide, the maximum moment occuring at the heavy wheel at the center of the span and the maximum shear with it at the end.

Equations 1 and 2 were used to determine the depth required by the given stresses in bending and in shear respectively. Computations show that the depth of the slab is governed by the moment and found to be equal to d = 5.75 in. and 1 3/4 in was added to this value in order to obtain the total depth. This provides $1\frac{1}{2}$ in of concrete below the steel to protect it from corrosion. The actual value of d, taken for the slab is so near to that theoretically necessary for balanced design, that the steel area was calculated without discernible error by equation 3,using the approximate value for j as 7/8. The operation of choosing the steel and its spacing consists in dividing the $\frac{1}{2}$ in. round bar area by the area required per inch thus determining the spacing for each size. The bond stress is found by the equation 4, and checks the design perfectly.

Computations

Assume 8" slab

18" x 12" beam

Clear span of slab = $8\frac{1}{2}$'

Ketchum's Specifications: $W = 2/3\ S + T$

$W = 2/3\ (10) + 1 = 7.68$ ft.

Dead Load

2" bituminous surfacing .... 25 lbs.

8" slab ....100 lbs.

Total Dead Load .... 125 lbs.

Dead Load Moment

$125\ (8.5)^2 \times 1/8 \times 8/10 = 905$ ft.lbs.

Live Load Moment

$\frac{12,000}{7.68} \times 8.5 \times 1/4 \times 8/10 = 2660$ ft. lbs.

Impact 30% $= 800$ ft.lbs.

Total moment $= 4365$ ft.lbs.

Shear

Live Load

$\frac{12,000}{7.68} \times 1/2 = 790$ lbs.

Impact 30% $= 238$ lbs.

Dead Load

$125 \times 8 \times 1/2 = 535$ lbs.

Total Shear $= 1563$ lbs.

Eq. 1 $d = \sqrt{\frac{M \times 12}{131 \times b}}$ Eq. 2 $d_v = \frac{V}{b \times 7/8 \times v}$

$$d_m = \sqrt{\frac{4365 \times 12}{131 \times 12}} = 5.75 \text{ in.}$$

$$d_v = \frac{1563}{12 \times 7/8 \times 40} = 3.69 \text{ in.}$$

Use slab $7\frac{1}{2}$" ($d = 5.75$ in)

Steel

Eq. 3 $\quad A_s = M/f_s jd$ $\qquad$ Eq. 4 $\quad u = \frac{vb}{\Sigma o}$

$$A_s = \frac{4365 \times 12}{20{,}000 \times 7/8 \times 5.75} = .52 \text{ sq. in./ ft. width}$$

$$= .0432 \text{ sq. in. / in. width}$$

Try $\frac{1}{2}$ in round bars $\quad A = .196$ sq. in.

$$\text{spacing } \frac{.196}{.0432} = 4.5 \text{ in}$$

$$v = \frac{V}{bjd} = \frac{1563}{12 \times 7/8 \times 5.75} = 26 \text{ lbs.}$$

$$u = \frac{26 \times 4.5}{1.57} = 75 \text{ lbs. (less than 100 lbs)}$$

Use Slab $7\frac{1}{2}$" $\quad$ (d= 5.75 in.)

Steel $\frac{1}{2}$" round bars $\quad$ Spacing $4\frac{1}{2}$"

INTERIOR BEAM DESIGN:

The method and procedure in the design of the interior beam of the floor system is given with the necessary calculations and figures. Fig C, in the next computation sheet shows the position of live loads and dead load for the calculation of the maximum moment and shear in the beam. Before the dead load stresses are known the size of the beam stem must be fixed, which may be done approximately by computations on scratch paper, giving a trial size to be used as the basis of furhter investigation. An 18" x 12" beam was assumed at the start, but further computations showed that it was neccessary to change the beam to 26" x 18". The method to find the depth of the beam is similar to that of the slab and the same equations are used; thus the the size of the beams were determined by the equations 1 and 2.

To find the steel to be used the same method is followed as in the slab. Equation 3 was used to find the area of steel required for the reinforcement of the beam. 6 square bars of area equaling to 1 sq. in. were used in the beams.

The design of these interior beams were completed by making a sketch to give all the essential information regarding the beams which is to serve as a basis for the design of the diagonal tension reinforcement. The bending up the steel and the stirrups were the last steps in the computations of the interior beam design.

The same method is used for the design of the exterior beams or the end beams. This completed the design of the interior and the exterior beams for the floor system.

Computations (Interior beam)

Wheel Load

4 x 15 x 2000 = 12,000 lbs. (rear wheel)

Dead Load

Surface 2"	25 lbs./sq.ft.
Slab $7\frac{1}{2}$"	94 lbs./sq.ft.
	119 lbs/sq.ft.

Dead Load = 119 x 10 = 1190 lbs./ft. of beam

Assume 18 x 12 beam
stem = 225 lbs./ ft of beam

Total Dead Load = 1415 lbs./ft. of beam

Moment

Dead Load Moment (positive)

1/8 (1415) x $\overline{18^2}$ x 12 = 687,500" lbs.

Live Load Moment (positive)

24,000 x 9 - 12,000 x 7.5 - 12,000 x 1.5 = 1,300,000" lbs.

Impact (pos.)

30% of 1,300,000 = 390,000" lbs.

Total Positive Moment = 2,377,500" lbs.

Total Negative Moment =

8/20 (2,377,500) = 950,000" lbs.

Shear

Live Load = 24,000 lbs.

Impact (30%) = 7,200 lbs.

Dead Load = $\frac{1}{2}$ x 1415 x 18 = 12,720 lbs.

Total Shear = 43,920 lbs.

Hand-book Specification for T Beam

$b = L/4$

$18/4 = 4.5$

Eq. 1

$$d_M \text{ (pos.)} = \sqrt{\frac{2,377,500}{131 \times 4.5 \times 12}} = 18.35''$$

Eq. 5 $$d_{-M} = \sqrt{\frac{M}{157 \times b}}$$

$$d_M \text{ (neg.)} = \sqrt{\frac{950,000}{151 \times 12}} = 22.50''$$

Eq. 2

$$d_v = \frac{43,920}{120 \times 7/8 \times 18} = 35.00''$$

Shear governs

Try $b = 18$

$$d_v = \frac{45,920}{120 \times 7/8 \times 18} = 23.2''$$

Corrections (Moment)

Live Load = 1,300,000 in.lbs.

Impact = 390,000 in.lbs.

Dead Load = 796,000 in.lbs.

Total M+ = 2,486,000 in.lbs. Total M- = 995,000 in.lbs.

Shear

Live Load = 24,000 lbs.

Impact = 7 ,200 lbs.

Dead Load = 14,750 lbs.

Total Shear = 45,950 lbs.

$$d_v = \frac{45,950}{120 \times 7/8 \times 18} = 24.3" \quad \text{use } d = 24"$$

Use 26" x 18"   (d = 24")

Steel

Eq. 3

$$\text{(Pos. M)} \quad A_s = \frac{2,486,000}{20000 \times 7/8 \times 24} = 5.95 \text{ sq.in.}$$

$$\text{(Neg. M)} \quad A_s = \frac{995,000}{20000 \times 7/8 \times 24} = 2.37 \text{ sq.in.}$$

Use 6 - 1" square bars

$\Sigma A = 6.0$ sq.in.

$\Sigma o = 24.0$ in.

Eq. 6 $u = V/\Sigma ojd$

$$\frac{45,950}{24 \times 7/8 \times 24} = 92 \text{ lbs. (less than 100)}$$

Bending up the Steel

Bend three bars at the shown distances as in(Fig. E)

$d_1 = 44$ in.

$d_2 = 62$ in.

$d_3 = 77$in.

Stirrups

$v = V/bjd$

$$v = \frac{45,950}{18 \times 7/8 \times 24} = 120 \text{ lbs.}$$

$v' = 120 - 40 = 80$ lbs.

Maximum Shear at Center Line

$$R_1 = \frac{(12,000 \times 3) + (12,000 \times 9)}{18} = 8000$$

$$V_{C.L.} = \frac{8000}{7/8 \times 24 \times 18} = 21.2 \text{ lbs.}$$

Use $\frac{1}{2}$" round stirrups

Maximum Spacing

$.6d = 24 \times .6 = 14.4$ in.

End Spacing

$S = 2 \times .1963 \times 16{,}000 = 6270$ lbs.

$S =$ End spacing x $v'b'$

Spacing $= 4.35$ use 4in.

Where 12" spacing occurs

$$v = \frac{6270}{12 \times 18} = 29 \text{ lbs.}$$

$80/29 = 88/x$

$x = 32$ in.

Stirrups and Their Spacing

Use $\frac{1}{2}$ round stirrups

Spacing:

1 @ 2 in.

5 @ 6 in.

6 @ 12 in.

Computations (End Beam Design)

Assume 26 x 14 beam.

Moment

Live Load Moment (pos.)	= 1,300,000"lbs.
Impact 30% (pos.)	= 390,000"lbs.
Dead Load Moment (pos.)	= 353,000"lbs.
Total Positive Moment	= 2,043,000"lbs.
Total Negatve Moment	= 820,000"lbs.

Shear

Live Load	= 24,000 lbs.
Impact 30%	= 7,200 lbs.
Dead Load	= 7,840 lbs.
Total Shear	= 39,040 lbs.

d_M (pos.)

$$= \sqrt{\frac{204,000}{131 \times 60}} = 16.4 \text{ in.}$$

d_M (neg.)

$$= \sqrt{\frac{820,000}{151 \times 1.4}} = 19.6 \text{ in.}$$

$$d_V = \frac{39,040}{120 \times 7/8 \times 14} = 26.6 \text{ in.}$$

Try 26" x 16" beam (d = 24")

Corrections

Dead Load = 8150 lbs.

Total Shear = 39,350 lbs.

$$d_V = \frac{39,350}{16 \times 7/8 \times 120} = 23.4" \quad \text{use } 24$$

Use 26" x 16" beam (d = 24")

Steel

$$(\text{Pos. M})\ A_s = \frac{2,056,000}{20,000 \times 7/8 \times 24} = 4.9 \text{ sq. in.}$$

$$(\text{Neg. M})\ A_s = \frac{825,000}{20,000 \times 7/8 \times 24} = 1.47 \text{ sq.in.}$$

Try 1 1/8" round bars

4 bars $\quad \Sigma o = 18 \text{in.}$
$\quad \Sigma A = 5.06 \text{ in}^2$

2 bars $\quad \Sigma o = 9 \text{ in.}$
$\quad \Sigma A = 2.53 \text{ in}^2$

Bond

$$u = \frac{39,500}{18 \times 7/8 \times 24} = \text{(all right) } 103 \text{ lbs.}$$

Use 1 1/8 round bars.

Bend Up

$d_1 = 54$ in.

$d2 = 77$ in.

Stirrups and Their Spacing

Use $\frac{1}{2}$ round stirrups

Spacing : 1 @ 2 in.

5 @ 6 in.

6 @ 12 in.

GIRDER DESIGN

The girders carry the concentrated loads of the beam reactions, plus their own load due to uniform dead load. They have been designed as continuous beams over the whole span of the bridges supported by columns at every 20 feet.

The moments in the girder have been found by the balancing process or the Hardy-Cross method as explained in the book " Theory of Modern Structures (Vol. II) ".

Fig. H shows the respective positions of the girder, column, and arch with their proper stiffness factor. The common idea of the stiffness is that it is a property which increases with the increase of cross sectional area and decreases with the increase of length. In this particular case, the definition of stiffness must be the resistance to rotation, because the unbalanced moment at a joint will be divided between the members meeting at the joint in proportion to their respective resistances to rotation or in inverse proportion to their respective end rotations under the effect of unit end moments. The columns have been designed to resist a total load of 80,848 lbs.

Computations

Loading (see fig. H)

Surfacing 25 lbs.

Slab = (94 + 25) x 10 = 1190 lbs.

26" x 18" beam = 346 lbs.

1536 lbs.

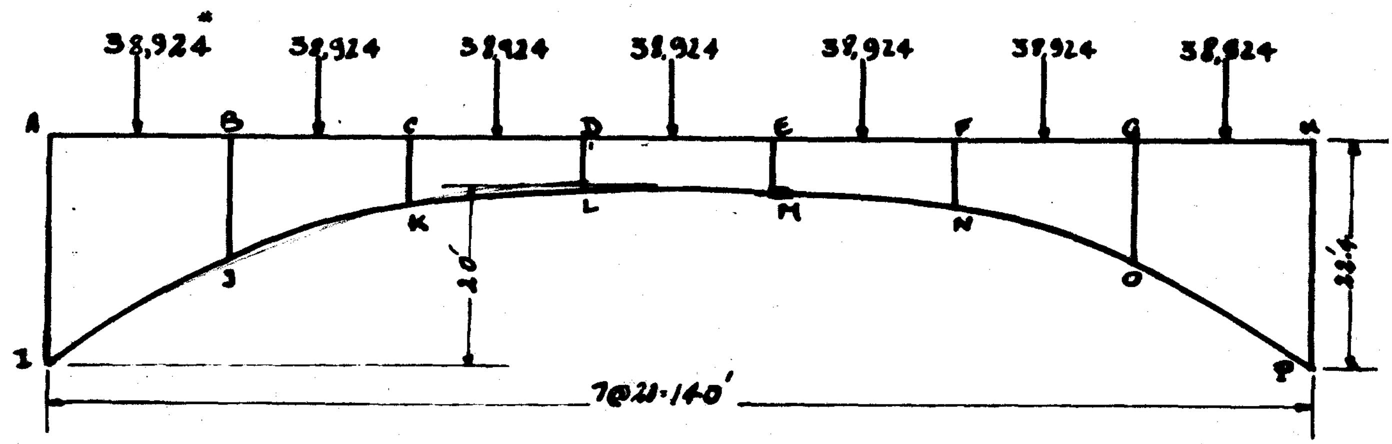


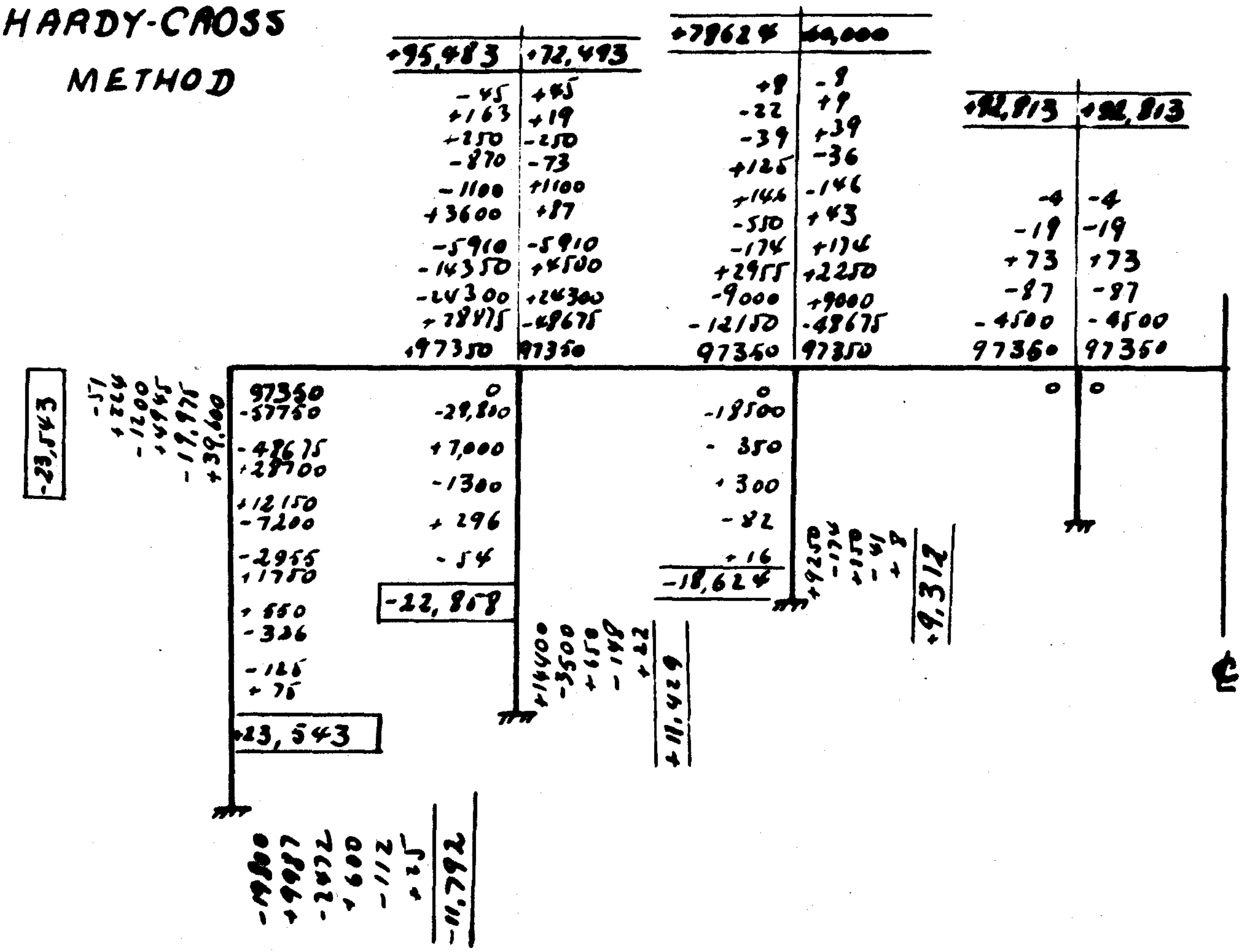
Fig. H

NOTE :

Stiffeness factors & length of mem.

AI	= 1	= 22.40'
BJ	= 1.73	= 12.88'
CK	= 3	= 7.28'
DL	= 4.6	= 4.80'
AB	= 1.46	= 20.00'

MOMENT DISTRIBUTION
BY
HARDY-CROSS
METHOD



Interior

1536 x 9 = = 13,824 lbs.

Curb 9/12 x 12/12 x 150 = 1,100 lbs.

14,924 lbs.

Exterior

930 x 9 = 8,127 lbs.

Curb = 500 lbs.

8,627 lbs.

Assume 26" x 18" girder

Inertia of Girder

$bd^3 = 18 \times (26)^3 = 26,400 \text{ in}^4$

Assume 12" x 12" @ 20' Column

Inertia of Column = 20,100 in^4

Moment (see fig. J and Table)

$M_{C.L._1}$ = 171,077 'lbs.

$M_{C.L._2}$ = 171,077 + 95,843 = 266,660 ' lbs.

$M_{C.L._3}$ = 329,240 'lbs.

$M_{C.L._4}$ = 296,427 'lbs.

Dead Load of Girder (assume) 346 lbs.

Dead Load Moment (pos.)

1/16 346 x 400 = 8,650 'lbs.

Dead Load Moment (neg.)

$1/12 \; wl^2$ = 11, 500 'lbs.

Maximum Positive Moment

329,240 + 8650 + (impact 30%) 98,600 = 436,490 'lbs.

Maximum Negative Moment = 135,583 'lbs.

Shear

Maximum Shear = 38,924 lbs.

$\frac{1}{8}wl^2$ = 3,460 lbs.

42,384 lbs.

$$d_m = \sqrt{\frac{436,490 \times 12}{131 \times 18}} = 47 \text{ in.}$$

$$d_v = \frac{42,384}{120 \times 7/8 \times 18} = 22.4 \text{ in.}$$

Try G-1 36" x 36" 1350 lbs.

Maximum Moment (pos.) = 329,240 'lbs.

Impact = 98,600 'lbs.

Dead Load Moment = 33,700 'lbs.

Total Moment = 461,540 'lbs.

$$d_m = \sqrt{\frac{461,540 \times 12}{131 \times 36}} = 34 \text{ in.}$$

Use 36" x 36" Girder (d = 34")

Corrections

Maximum Positive Moment = 461,540 'lbs.

Maximum Negative Moment = 169,083 'lbs.

Maximum Shear = 52,424 lbs.

Steel

(pos. M.) $A_s = \frac{461,540 \times 12}{20,000 \times 7/8 \times 34}$ = (9.3 sq.in.

(neg. M.) A_s = 3.41 sq.in.

Use 6 1 1/4 Square Bars.

Bond = $\frac{52,424}{30 \times 7/8 \times 34}$ = 60 lbs. (less than 100)

Bending Up (see fig. L)

$d_1 = 4.1'$

$d_2 = 5.8'$

Stirrups

Use $\frac{1}{2}$ Round Stirrups

Spacing 1 @ 2"

5 @ 6"

7 @ 12"

COLUMN DESIGN

Computations

Total Loading	38,924 lbs.
	14,924 lbs.
	27,000 lbs.
	= 80,848 lbs.

Bending Moment = 23,540 'lbs.

2000 lbs. $f_c = 450$ lbs.

$$A_g = \frac{80,848}{450} = 180 \text{ sq.in.}$$

Try 12"x 12" A = 144 sq.in.

180 - 144 = 36 sq.in.

$$A_s = \frac{36}{n-1} = \frac{36}{14} = 2.5 \text{ sq.in.}$$

$$\frac{100 \times 2.5}{144} = 1.74\ \%$$ (limit $\frac{1}{2}$ - 2 %) All right

Use 4 - 1" Round A = 3.14 in^2

Area of 12" x 12" = 144

Area of 3.14 x 14 = 44

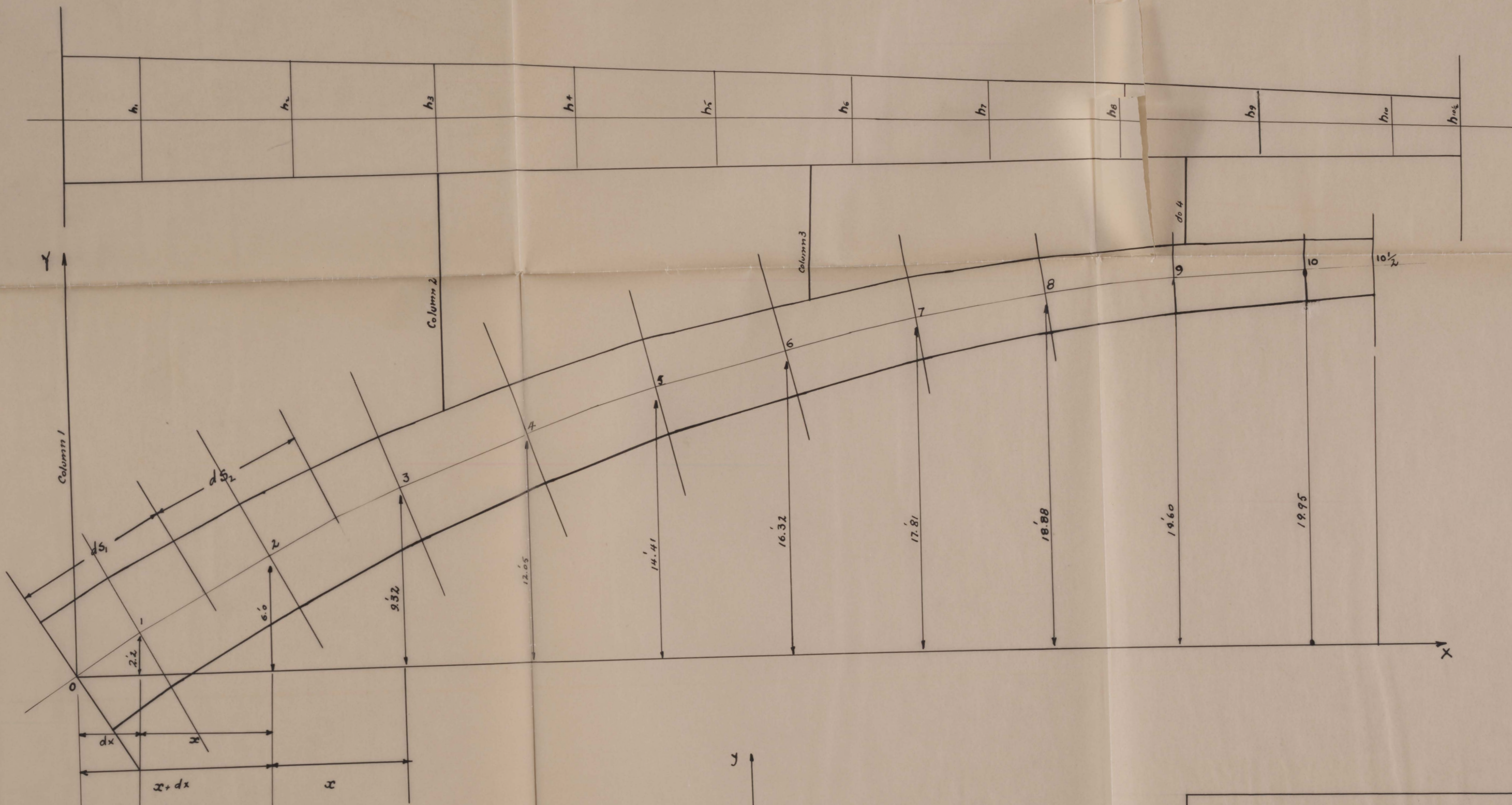
$$144 + 44 = 188 \text{ sq.in.} \quad (\text{ greater than } A_g = 180 \text{ })$$

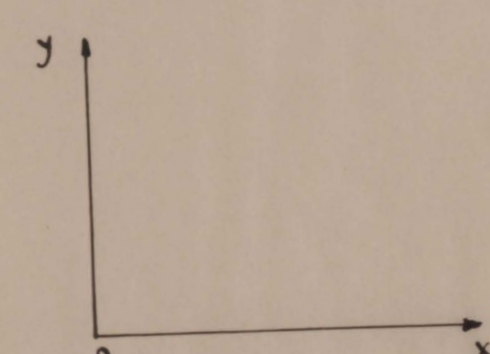
Use Column

12" x 12"

4 - 1" Round

$\frac{1}{4}$ Round Ties @ 12"





Drawing for Computing the DIFFERENT THICKNESS OF THE ARCH at different Points

Drawn By : Jorgi Pavlidis	Drawing Nº 1
Scale 1" = 4' Date 1939	

ARCH DESIGN

To design the arch the elastic theory was used the method being the one used by the Bureau of Public Roads. As this is more complicated and different compared with the other parts of the design it is neceessary to explain the method and its theory.

A hingeless arc is statically indeterminate, because looking at Fig. N we see that $H_o = H_x$, $V_x = V_0 - P$, and $M_x = M_o + V_o x - P(x - a) - H_o y$. This gives only three equations to find the six unknowns which makes the arc indeterminate, and naturally three additional equations are required. These are derived from the elastic properties of the arch. The derivation can be found in Analysis of Concrete Arches, reprint from the publication of the Bureau of Public Roads "Public Roads".

Arch computations are handled best on tabulated forms as shown on the prints. These require that a preliminary arch be worked and the stresses in it computed. These stresses are computed from moments, shears, and the thrusts at points 5, 3, 8, and 10.5 of the arch ring.

Changes in temperature in the arch ring produce stresses and moments that are either positive or negative depending on whether the temperature is decreasing or increasing. The suggested changes used here are, a rise of 30° F. and a fall of 40° F. The formulas for computing the temperature effect are shown in the prints.

The Bureau of Public Roads does not suggest a method of determining the method for determining the size and shape of the trial arch and the following method has been used:

The crown is the highest point of the arch and a trial thickness is computed by the following formula: $h = .001\ (11{,}000 + l^2)$ which is taken from the book " Reinforced Concrete Construction" .

Computations

Size & Shape of Trial Arch

$$y = b - \frac{8rL}{6 + 5r}(3c^2 + 10c^4 r)$$

where

b = Rise

$r = b/L$

$c = .5 - x/L$

Using these formulas the following answers were found for the 'y' s.

y_a = 7.67 ft.
y_b = 13.30 ft.
y_c = 17.09 ft.
y_d = 19.28 ft.
y_1 = 2.20 ft.
y_2 = 6.00 ft.
y_3 = 9.32 ft.
y_4 = 12.05 ft.
y_5 = 14.41 ft.
y_6 = 16.32 ft.
y_7 = 17.81 ft.
y_8 = 18.88 ft.
y_9 = 19.60 ft.
y_{10} = 19.95 ft.

Knowing the abscissa and the ordinates (x , y), it is possible to plot the axis of the arch.

The rest of the computations of the arch are tabulated and made in print form. This completed the design of the reinforced concrete bridge.

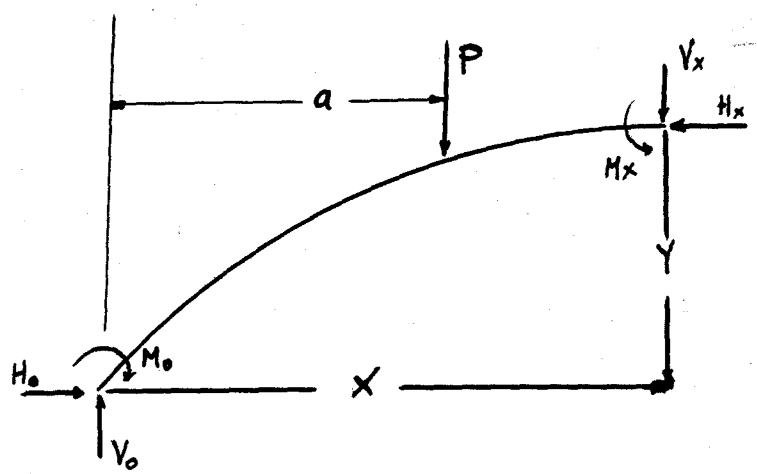


fig. N

Table 1 Computations of Δ

1	2	3	4	5	6	7	8	9	10	11
Points	$h = A$	h^3	$I_c = \frac{h^3}{12}$	$\frac{h}{2}$	$\frac{h}{2} - d'$	$\left[\frac{h}{2} - d'\right]^2$	$I_s = 2\left[\frac{h}{2} - d'\right]^2 A_s$	$I = I_c + I_s$	ds	$\Delta = \frac{ds}{I}$
0	6.8	315	26.3	3.40	3.23	10.40	6.75	33.05		
1	6.6	[illegible]	24.0	3.30	3.13	[illegible]	6.38	30.38	8.20	.270
2	[illegible]	[illegible]	19.8	[illegible]	[illegible]	[illegible]	5.60	25.40	8.00	.315
3	[illegible]	212	127	[illegible]	[illegible]	7.35	4.78	[illegible]	7.64	.340
4	[illegible]	[illegible]	[illegible]	[illegible]	2.61	6.00	4.40	11.73	7.56	.406
5	[illegible]	140	11.7	2.60	2.43	5.90	3.84	15.64	7.36	.475
6	[illegible]	110	[illegible]	2.40	[illegible]	4.92	3.24	12.44	7.26	.582
7	4.4	85	7.1	2.20	2.03	4.12	2.68	9.78	7.22	.740
8	4.0	64	5.3	2.00	1.83	3.35	2.18	7.48	7.20	.964
9	3.6	47	3.9	1.80	1.63	2.66	1.73	5.63	7.12	1.265
10	3.36	38	3.2	1.68	1.51	2.28	1.48	4.68	7.12	1.525
									$\frac{1}{2}\Sigma\Delta$	6.001

Table 2 Computations of V_0

12	13	14	15	16	17	18
$z - 20$, $z = \frac{2x}{dx}$	$a = (z-20)\Delta$	$-\sum_a^c (z-20)\Delta$	$\frac{1}{2}\sum_a^c \left(z - \frac{2x}{dx}\right) a$	$Z^2(40-Z)^2$	$\left[Z^2(40-Z)^2\right]\Delta$	V_0
-19	-5.13	5.13	[illegible]	[illegible]	[illegible]	[illegible]
-17	-5.35	10.48	[illegible]	1378	421	1.000
-15	-5.10	15.58	[illegible]	1250	425	[illegible]
-13	-5.26	20.84	480.12	[illegible]	461	.950
-11	-5.22	26.06	[illegible]	[illegible]	[illegible]	[illegible]
-9	-5.25	31.31	433.[illegible]	[illegible]	[illegible]	.855
-7	-5.18	36.49	[illegible]	[illegible]	[illegible]	[illegible]
-5	-4.83	41.32	366.42	860	[illegible]	[illegible]
-3	-3.80	46.12	324.10	[illegible]	1035	[illegible]
-1	-1.53	46.66	[illegible]	[illegible]	1225	.560
	-46.65	278.98		$\Sigma\Delta Z^2$	6518	

Table 3 Computations of H_0

19	20	21	22	23	24	25	26	27	28	29	30
Points	Y	Δ	$Y\Delta$	$Y - \frac{\Sigma Y\Delta}{\Sigma\Delta}$	$R = \Delta\left(Y - \frac{\Sigma Y\Delta}{\Sigma\Delta}\right)$	$\sum_a^c \Delta\left(Y - \frac{\Sigma Y\Delta}{\Sigma\Delta}\right)$	$\frac{1}{dx}\sum_a^c \left(z - \frac{2x}{dx}\right) R$	$Y\Delta\left(Y - \frac{\Sigma Y\Delta}{\Sigma\Delta}\right)$	$\cos\phi$	$\frac{\cos\phi}{A}$	H_0
0	[illegible]	[illegible]	[illegible]	-16.50					.860	.126	
1	2.20	.270	.594	-14.30	-3.86	-3.86	0	-8.50	.861	.130	0
2	6.00	.315	1.890	-10.50	-3.31	-7.17	-3.86	-19.85	.877	.141	.128
3	9.32	.340	3.170	-7.18	-2.46	-9.63	-11.03	-22.90	.916	.153	.374
4	12.05	.405	4.880	-4.45	-1.80	-11.43	-20.66	-21.70	.926	.166	.685
5	14.41	.475	6.850	-2.09	-.99	-12.42	-32.09	-14.30	.962	.183	1.061
6	16.32	.582	9.500	-0.18	-.10	-12.52	-44.51	-1.63	.970	.202	1.475
7	17.81	.740	13.200	1.31	.97	-11.55	-57.03	17.30	.978	.222	1.890
8	18.85	.964	18.200	2.38	2.30	-9.25	-68.58	43.40	.982	.246	2.270
9	19.60	1.265	24.800	3.10	3.93	-5.32	-77.83	76.95	.997	.277	2.580
10	19.95	1.525	30.500	3.45	5.26	-.06	-83.15	105.00	1.000	.298	2.760
	$\frac{1}{2}\Sigma\Delta Y$		113.584		-.06	-83.21	$\frac{1}{2}\Sigma =$	153.77	$\frac{1}{2}\Sigma =$	2.136	

$$F = \frac{1}{2}\Sigma Z^2\Delta - 200\Sigma\Delta = 506.6$$

$$C = \frac{1}{dx}\sum_a^c Y\Delta\left(Y - \frac{\Sigma Y\Delta}{\Sigma\Delta}\right) + \Sigma\frac{\cos\phi}{A} = 30.2$$

$$H_t = \frac{20etE}{C} = \left.\begin{matrix} 34{,}300 \\ -45{,}600 \end{matrix}\right\} \begin{matrix} 30^\circ \\ -40^\circ \end{matrix}$$

Span $l = 140'$ Rise $= 20'$

$dx = \frac{L}{20} = 7'$ $Z = \frac{2x}{dx}$

$\Sigma\frac{Y\Delta}{\Delta} = 16.5$

Table 4 Computations for M_o

31	32	33	34	35	36	37	38	39	40	41
	Z	H_o	V_o	Δ	$\sum_a^i \Delta$	$\frac{1}{2}\sum_a^i (Z-\frac{2a}{dx})\Delta$	$\frac{dx}{\Sigma\Delta}$(Col.37)	$\frac{\Sigma Y\Delta}{\Sigma\Delta} H_o$	$-20\frac{dx}{2}V_o$	M_o
0					13.742					
1	1	0	1.000	.270	13.472	130.599	75.600	0	-70.60	5.00
2	3	.128	1.000	.315	13.157	117.127	67.900	2.11	-70.00	.01
3	5	.374	.978	.340	12.817	103.970	60.250	6.16	-68.50	-2.14
4	7	.685	.940	.405	12.412	91.153	52.900	11.30	-66.50	-3.30
5	9	1.061	.905	.475	11.937	78.741	45.600	17.50	-63.40	-.30
6	11	1.475	.865	.582	11.355	66.804	38.700	24.30	-59.90	3.10
7	13	[illegible]	.791	.740	10.615	55.449	32.100	31.20	-55.40	7.90
8	15	2.270	.720	.964	9.679	44.894	26.000	37.40	-49.65	13.75
9	17	2.584	.640	1.265	8.406	35.168	20.400	42.50	-44.80	18.10
10	19	2.760	.550	1.525	6.881	26.757	15.000	45.50	-38.50	22.50
10'	19	2.760	.450	1.525	6.356	19.876	11.500	45.50	-31.50	25.50
9'	17	2.584	.360	1.265	4.091	14.620	8.420	42.50	-25.20	26.72
8'	15	2.270	.280	.964	3.127	10.429	6.050	37.40	-19.60	23.85
7'	13	1.890	.209	.740	2.387	7.302	4.050	31.20	-14.65	20.80
6'	11	1.475	.145	.582	1.806	4.915	2.850	24.30	-10.01	17.14
5'	9	1.061	.095	.475	1.330	3.110	1.800	17.50	-6.65	12.65
4'	7	.685	.050	.405	.925	1.780	1.030	11.30	-3.50	8.83
3'	5	.374	.022	.340	.585	.855	.496	6.16	-1.54	5.11
2'	3	.128	.00	.315	.270	.270	.157	2.11		2.26
1'	1	0	.00	.270	0	0	0	0		
0'	0									

V_o for right half found by subtracting values for left half from 1.

$V'_{o7} = 1 - V_{o7} = 1 - .791 = .209$

$$H_o = \frac{dx}{\Sigma\Delta}\frac{1}{2}\sum_a^i \left(Z - \frac{2a}{dx}\right)\Delta + H_o\frac{\Sigma Y\Delta}{\Sigma\Delta} - 20\frac{dx}{2}V_o$$

$\frac{dx}{\Sigma\Delta} = .58$

$-20\frac{dx}{2} = -70$

For checks:

$\frac{\frac{1}{2}\text{ Sum of 22}}{\frac{1}{2}\text{ Sum of 11}}$ Sum of 33 = Sum 39

435 = 436

$-20\frac{dx}{2}$ (Sum of 34) = Sum 40

700 = 700

Sum 38 + 39 + 40 = Sum 41

207.4 = 207.4

Computation of Dead Loads

Pt.	bds	150 bds	D.L
1	54.10	8110	8110
2	49.60	7440	7440
3	45.60	6840	6840
4	42.00	6300	6300
5	38.30	5690	5690
6	34.70	5120	5130
7	31.80	4765	4765
8	[illegible]	4210	4810
9	25.70	3850	3850
10	24.00	3600	3600

Table 5 Computation of M_3. Table 6 Computation of M_8 Table 7 Computation of $M_{10\frac{1}{2}}$

42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60
				Point 3 $Z=5$ $Y=9.32$					Point 8. $Z=15$ $Y=18.88$					Point $10\frac{1}{2}$ $Z=20$ $Y=20$				
	H_o	V_o	M_o	$V_o Z_3$	$V_o Z_3 - (Z_3 - \frac{2a}{dx})$	m_3	$-H_o Y_3$	M_3	$V_o Z_8$	$V_o Z_8 - (Z_8 - \frac{2a}{dx})$	m_8	$-H_o Y_8$	M_8	$V_o Z_{10\frac{1}{2}}$	$V_o Z_{10\frac{1}{2}} (Z_{10\frac{1}{2}} - \frac{2a}{dx})$	$m_{10\frac{1}{2}}$	$-H_o Y_{10\frac{1}{2}}$	$M_{10\frac{1}{2}}$
0																		
1	0	1.000	5.00	5.00	1.00	3.50	0	8.50	15.00	1.00	3.50	0	8.50	20.00	1.00	3.50	0	8.50
2	.128	1.000	.01	5.00	3.00	10.50	-1.194	9.32	15.00	3.00	10.50	-2.41	8.10	20.00	3.00	10.50	-2.56	7.95
3	.374	.978	-2.14			17.10	-3.490	11.47	14.65	4.65	16.30	-7.05	7.11	19.55	4.55	15.92	-7.47	6.31
4	.685	.950	-2.30			16.60	-6.300	7.91	14.25	6.25	21.90	-12.90	6.70	19.00	6.00	21.00	-13.70	5.00
5	1.061	.905	-.30			15.85	-9.880	5.67	13.58	7.58	26.50	-20.00	6.20	18.10	7.10	24.90	-21.22	3.38
6	1.475	.855	3.10			14.95	-13.750	4.30	12.68	8.68	30.40	-27.80	5.70	17.10	8.10	28.40	-29.50	2.00
7	1.890	.791	7.90			13.85	-17.600	4.16	11.86	9.86	34.55	-35.65	6.80	15.82	8.82	30.90	-37.80	1.00
8	2.270	.720	13.75			12.60	-21.200	4.65			37.80	-42.80	8.75	14.40	9.40	32.90	-45.40	1.25
9	2.580	.640	18.10			11.20	-24.000	5.30			33.60	-48.65	3.05	12.80	9.80	34.30	-51.16	1.24
10	2.760	.550	22.50			9.62	-25.800	6.32			29.20	-52.05	-.35	11.00	10.00	35.00	-55.20	2.30
10'	2.760	.450	25.50			7.87	-25.000	8.37			23.60	-52.05	-2.95			31.50	-55.10	1.90
9'	2.580	.360	25.72			6.30	-24.000	8.02			18.90	-48.65	-4.03			25.20	-51.55	-.63
8'	2.270	.280	23.85			4.90	-21.200	7.55			14.70	-42.80	-4.25			19.60	-45.40	-1.95
7'	1.890	.209	20.80			3.66	-17.600	6.86			10.90	-35.65	-3.95			14.60	-37.80	-2.40
6'	1.475	.145	17.14			2.54	-13.750	5.93			7.61	-27.80	-3.05			10.15	-29.50	-2.21
5'	1.061	.095	12.65			1.66	-9.850	4.43			4.99	-20.00	-2.36			6.65	-21.22	-1.92
4	.685	.050	8.83			.875	-6.390	3.31			2.63	-12.90	-1.44			3.50	-13.70	-1.37
3'	.685	.022	5.11			.385	-3.490	2.01			1.15	-7.05	-.79			1.54	-7.47	-.82
2'	.374	0	2.26			0	-1.194	1.07			0	-2.41	-.15			0	-2.56	-.30
1'	.128	0	0			0		0			0	0	0			0	0	0
0'	0											0	0					

$M_x = M_o + m_x - H_o Y$

$m_x = \left[V_o Z_x - \left(Z_x - \frac{2a}{dx}\right)\right] \frac{dx}{2}$ when $Z > \frac{2a}{dx}$

$m_x = V_o Z_x \frac{dx}{2}$ when $Z < \frac{2a}{dx}$

For checks:

Sum of 45 + 48 + 49 = Sum of 50

207.48 + 153.96 − 246.6 = 115

Sum of 45 + 53 + 54 = Sum of 55

207.48 + 207.48 − 498.6 = 37.6

Sum of 45 + 58 + 59 = Sum of 60

207.48 + 360.06 − 528.31 = 29.23

Table 8 Computations of H, V and M for Dead Loads

61	62	63	64	65	66	67	68	69	70	71	72	73	74
	Unit Loads						Dead Load		Dead Loads				
	H_o	V_o	M_o	M_3	M_8	M_{max}		H_o	V_o	M_o	M_3	M_8	M_{max}
1	0	1.000	5.00	8.50	8.50	8.50	8110	0	8,110	40,550	69,000	69,000	69,000
2	.128	1.000	.01	9.32	8.10	7.95	7440	.952	7,440	74	69,300	[illegible]	58,050
3	.374	.978	-2.14	11.47	7.11	6.31	6840	2,560	6,680	-14,610	78,400	48,600	43,100
4	.685	.950	-2.30	7.91	6.70	5.09	6300	4,320	5,990	-14,500	[illegible]	42,250	31,500
5	1.061	.905	-.30	5.67	6.20	3.38	5690	6,040	5,145	-1,705	32,300	26,300	[illegible]
6	1.475	.855	3.10	4.30	5.70	2.00	5130	7,560	4,390	15,900	22,100	29250	10250
7	1.890	.791	7.90	4.15	6.80	1.00	4,765	9,000	3,775	37,700	19,800	32450	4765
8	2.270	.720	13.75	4.65	6.75	1.85	4310	9,770	3,105	53,350	[illegible]	37800	[illegible]
9	2.580	.640	18.10	5.30	3.05	1.24	3850	9,940	2,470	69,750	[illegible]	11 750	[illegible]
10	2.760	.550	22.50	6.32	-.35	2.30	3600	9,920	1,980	81,000	22,750	-1260	[illegible]
10′	2.760	.450	25.30	8.59	-2.95	1.90	3600	9,920	1,620	91,750	30100	-10,610	6,840
9′	2.580	.360	25.72	8.02	-4.03	-.63	3850	9,940	1,875	99,000	30900	-15,500	-2,430
8′	2.270	.280	23.85	7.55	-4.25	-1.93	4,310	9,770	1,207	[illegible]	[illegible]	-18350	-8325
7′	1.890	.209	20.80	6.86	-3.95	-2.40	5130	7560	995	99,150	32700	[illegible]	-11450
6′	1.475	.145	17.14	5.93	-3.05	-2.21	5690	6040	745	[illegible]	32400	-15650	[illegible]
5′	1.061	.095	12.66	4.43	-2.36	-1.92	6300	6040	540	71950	25150	-13400	[illegible]
4′	.685	.050	8.83	3.31	-1.44	-1.37	6840	4320	347	55650	20900	-9675	[illegible]
3′	.374	.022	5.11	2.01	-.79	-.82	7440	2560	150	35070	13750	-5400	[illegible]
2′	.128	0	2.26	1.07	-.15	-.30	8110	952	0	16800	7,960	-1130	-2230
1′	0	0	0	0	0	0	8110	0	0	0	0	0	0
	26.446	10.000	212.22		60.91	40.83	112,070	120,124	56,564	964,624		366,600	[illegible]
			-4.74		-22.61	-11.58				-30,815		-109,225	60,920
			207.48	115.14	38.29	29.25				933,809	[illegible]	257,375	201,424

Temp. Mom.

$$H_t = -H_t\left(Y - \frac{\Sigma Y\Delta}{\Sigma\Delta}\right)$$

For H_t see page 1

$Y - \frac{\Sigma Y\Delta}{\Sigma\Delta}$ from Col. 23

Point				
0	H_t = +566,000		8	H_t = −81,550
	−752,000			+108,500
3	H_t = +246,000		$10\frac{1}{2}$	H_t = −120,000
	−327,000			+159,800

Table 9 Computation of Maximum Stresses

75	76	77	78	79	80	81	82	83
		H	M	V	$H\cos\phi$	$V\sin\phi$	N	Computation of Unit Stresses, f_c and f_s
Point 0	DL	120124	-933,809	56564	104250	28900	133150	- Mom. shows stress is max. in Extrados X_o = eccentricity = 5.03 $p = .019$ Use Dia 5 $\frac{N x_o}{f_c b h^2} = .25$ $\frac{t}{x_o} = \frac{h}{x_o} = 1.2$ $f_c = 1200$ $f_s = n f_c \left(\frac{h-d'}{kh} - 1\right) = 13000$ $K = .56$
	+CLL	112,500	428000	[illegible]	96700	[illegible]	119700	
	+CDL	358000	1525000	[illegible]	308000	[illegible]	408000	
$\sin\phi = .510$	-CLL	112500	-3,870,000	11850	96700	6040	102740	
$\cos\phi = .860$	-CDL	358000	-1,075,000	42700	308000	21500	329500	
$h = 6.8$	+T	34300	568,000		29500		29500	
	-T	-45600	-752000		-39200		-39200	
	-M		-5105809				1012590	
	+M		1581191				693500	
Point 3	DL	120124	-628000	41,014	110000	16,800	126,800	+ Mom. shows max. stress in Intrados $x_o = 2.23$ $p = .023$ $\frac{h}{x_o} = 2.68$ $\frac{N x_o}{f_c b h^2} = .3$ $f_c = 990$ $f_s = 14,900$ $K = .490$ Use Dia. 5
	+CLL	200,000	830,000	145000	185000	28000	213000	
	+CDL	720,000	2980000	68000	670,000	101000	771000	
$\sin\phi = .4101$	-CLL							
$\cos\phi = .916$	-CDL							
$h = 5.96$	+T	34300	246000		31,400		31400	
	-T	-45600	-327000		-41800		-41800	
	-M							
	+M		3101000				1391200	
Point 8	DL	120124	-257375	12,284	118050	2305	120355	Extrados: - Mom. $X_o = 0$ $p = .019$ Dia. 5 $\frac{h}{x_o} = 2.16$ $\frac{N x_o}{f_c b h^2} = /$ $f_c = /$ $f_s = /$ $K = /$ Intrados: + Mom. $X_o = 1.27$ $\frac{h}{x_o} = 1.15$ $\frac{N x_o}{f_c b h^2} = .18$ $f_c = 1000$ $f_s = 1500$ $k = .85$ Diag. 5
	+CLL	100000	357000	55600	98200	10450	98650	
	+CDL	357800	1282000	200,000	357000	37600	394600	
$\sin\phi = .185$	-CLL	102000	-177500	11750	100000	2210	102210	
$\cos\phi = .982$	-CDL	365500	-636500	42650	359000	8020	367020	
$h = 4.0$	+T	34300	-81550		33700		33700	
	-T	-45600	+108500		-44400		-44400	
	-M		0				763,075	
	+M		857625				671445	
Point $10\frac{L}{2}$	DL	120124	-201424	0	120124	0	120124	Extrados: - Mom. $X_o = 0$ $p = .0042$ Dia. 5 $\frac{N x_o}{f_c b h^2} = /$ $f_c = /$ $K = /$ $f_s = /$ Intrados: + Mom. $X_o = .87$ $\frac{N x_o}{f_c b h^2} = .35$ $f_c = 1200$ $K = .90$ $f_s = 13000$ Dia. 5
	+CLL	100000	274000		274000		274000	
	+CDL	357000	984000		357000		357000	
$\sin\phi = 0$	-CLL	100000	-82000		100000		100000	
$\cos\phi = 1$	-CDL	357000	-297000		357000		357000	
$h = 3.1$	+T	34300	-120000	0	34300		34300	
	-T	-45600	+159800		-45600		-45600	
	-M							
	+M		920400				1062524	

Explanation of Tables on Prints

The value of h_x for each of the ten points between 1 and 10 are computed by the formula and entered in

Col. 2 of table 1 of prints.

Col. 3 -5 Follow the table

Col. 6 d' = Fireproofing or steel cover =2" = .17 ft.

Col. 8 n = 10, 12 or 15 according to the value of f_c'used.

A_s = Total area of steel per ft. of arch ring in sq.ft.

Usually 1", 1 1/8" or 1½" Square bars are used, one per ft. at both top and bottom of the arch ring.

Col. 9 Add values in Col. 4 and in Col. 8.

Col. 10 ds = length of arch axis for each arch division scaled from drawing. Ds is not constant as dx is. See Pt. 3, Drawing I.

Col. 11 Sum of this Col. = ($\frac{1}{2}$) , as in this column only one-half of the arch ring is used.

Col. 12 dx = .05L. The values of this column are the same for all the arches.

Col. 13 Multiply the values in Col. 12 by those in Col. 11. Add Column.

Col. 14 Start at Pt. 1 with value as Col. 13, with the sign changed. (89.3) Prints. Add 89.3 and 171.7 = 261.0 for value at Pt.2, 261.0 and 243.0 = 504.0 for Pt. 3. Continue this method and add

Col.15 column.

Col. 15 Value at Pt. 10 = Sum of Col. 14. Value at Pt. 9 = value at Pt. 10 in Col. 15 plus value of Pt. 9 in Col. 14. Value at Pt. 8 = value at Pt. 9 in Col. in Col. 15 plus value at Pt. 8 in Col. 14. Continue this process to top of Col.

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Since Pt. 1 is not far from O and since V should be very nearly 1 for a unit load at Pt. 1, the value of F from the formula should be very nearly equal to the value at Pt. 1 in Col. 15. In computed arch it is equal.

Col. 16 Follow the table of the computed arch. Same for all arches.

Col. 17 Products of values in Cols. 16 and 11. Sum of this column = z^2 , as Col. 16 gives the entire values of the arch and not for one half of the arch.

Col. 18 Compute F by formula. Divide values in Col. 15 by F.

Col. 20 Values of y are computed by formula for y.

Col. 21 Same as col. 11.

Col. 22 Products of values in Col. 20 by those in Col. 21. Add column to obtain $(\frac{1}{2})$ y .

$\Sigma y\Delta/\Sigma\Delta$ = Sum of Col. 22 divided by sum of Col. 11.

Col. 23 Subtract this value from each value of y in Col. 20.

Col. 24 Products of values in Col. 21 by those in Col. 23. As a check the sum of this column should be equal to zero.

Col. 25 Obtained from Col. 24 in the same manner as Col. 14 was obtained from Col. 13. Add this column.

Col. 26 At Pt. 1 value is O. At Pt. 2 value is the value of the point. 1 in Col. 26 plus the value of point 1 at Col. 25. At Pt. 3 the value is the value at Col. 26 at Pt. 2 plus the value in Col. 25 at Pt. 2. Continue this method. All signs are negative.

Col. 27 Values are the products of values in Col. 20 by those in Col. 24. Add for $\frac{1}{2}\Sigma y\Delta(y - \Sigma y\Delta/\Sigma\Delta)$.

Col. 28 Compute ϕ by formula for tan-Θ or scale from drawing. ϕ is the angle between the horizontal and the tangent between the arch axis at the point. Look up cos ϕ.

Col. 29 Divide values in Col. 28 by those in Col. 2. A = cross-section area of the arch ring. Since the ring that is designed is 1 ft. wide, h = A. Add Col. for $\frac{1}{2}$ cosϕ / A.

Col. 30 Compute C by formula.

$C = 1/.05L$ times twice the sum of Col. 27 plus twice the sum of Col. 29. H_0 = Values in Col. 26 divided by C. Compute H_t by $H_t = 34560t/C$. See print.

Col. 32 To obtain Z, subtract .5 from the point number and multiply by 2. As at Pt. 10, Z = (10 - .5)2 = 19.0 . Also for Pt. 10'.

Col 33 Same as Col. 3 for Pts. for 1 to 10 and Pts. 1' to 10'.

Col. 34 Same as Col. 18 for Pts. 1 to 10. Values between 1' and 10' are found by subtracting the corresponding values for Pts. between 1 and 10 from 1. At Pt. 7' value = 1-79915.

Col. 35 Same as Col. 11 for Pts. between 1 and 10, and 10' to 1'.

Col. 36 Value at Pt. 1' is 0. Value at 2' is the sum of the values at Pt. 1' in Col. 35 and Pt. 1' in Col. 36. Value at 3' is the sum of the values at Pt. 2' in Col. 35 and Pt. 2' in Col. 36. Continue this process.

Col. 37 Value at Pt. 1' is 0. Value at Pt. 2' = Value at Pt. 1' in Col. 37 and value at Pt. 2' in Col. 36. Value at Pt. 3' = Value at Pt. 2' in Col. 37 and value at Pt. 3' in Col. 36. Continue this method.

Col. 38 Compute $dx/\Sigma\Delta . \Sigma\Delta$ twice the sum of Col. 11. Multiply each of the values in Col. 37 by this constant.

Col. 39 Multiply values in Col. 33 by $\Sigma y\Delta/\Sigma\Delta$ as computed for use in Col. 23.

Col. 40 Compute -20dx/2 and multiply values in Col. 34 by it.

Col. 41 Value for each Pt. in this Col. = Sum of the values for the same point in Cols. 38, 39 and 40.

Col. 43, 44, and 45. Same as Cols. 33, 34, and 35.

Col. 46 $Z_3 = 2x_3/dx = 2(7.5)/3 = 5.$ $y_3 = 3.51.$ Compute V_0Z_3 for points up to and including Pt. No.2.

Col. 47 $a_1 = x_1$, $a_2 = x_2$, etc. For Pt. 1 $Z_3 - 2a/dx = 5 - 2(1.5)/3 = 4.$ For Pt. 2, $Z_3 - 2a/dx = 5 - 2(4.5)/3 = 2$, etc.

Subtract these values from the corresponding values of Col. 46

Compute up to and including Pt. No. 2.

Col. 48. For Pts. up to but not including Pt. No. 3 multiply values in Col. 47 by $dx/2 = 3/2 = 1.5$. For Pt. 3 and beyond multiply the values ~~by~~ of V_0 in Col. 44 by $Z_3dx/2 = 5(3)/2 = 7.5.$

Col. 49 Multiply values of H_0 in Col. 43 by $-y_3$.

Col. 50 Value at any point = Sum of values for same point in Cols. 47, 48 and 49. See formula on Print 4.

Cols. 51, 52, 53, 54 and 55 are computed in similar manner to Cols. 46, 47, 48, 49 and 50.

Cols. 56, 57, 58, 59 and 60 are computed in a manner similar to Cols. 46, 47, 48, 49 and 50. Carry to Pt. 10 inclusive Cols. 56 and 57. For checks use equations on Print.

Cols. 61, 62, 63, 64, 65, 66 and 67. Copy Cols. 43, 44, 45, 50, 50, 55, 58 and 60.

Col. 68 Dead loads are really the weights of the archring in Open Spandril Arches and the weight of the arch ring plus the weight of the fill for the Filled Spandril Arches. The dead load at each point is the weight of the section of the arch ring and fill ds long. In the dead load table on Print, only Cols. 1, 2 and 3 are needed for open spandril arches.

Col. 69 to 74 Values are obtained by multiplying values in Cols. 62 to 67 by values in Col. 68. Sum of Col. 69 = H for dead load and is entered on table 9 Col. 77 as shown. Sum of Col. 70 = V for D. L. at Pt. 0 as shown, and is entered on table 9 at Pt. 0. Sums of Cols. 71, 72, 73 and 74 are the dead load moments for Pts. 0, 3, 8 and 10.5 and are entered on table 9 in Col. 78 as shown.

Temperature Moments and Thrusts

These are entered on table 9 as positive and negative as shown. Temperature V is always equal tozero. Temperature H is the same for all points and is computed as shown before, both positive and negative being entered on table 9. Temperature moments for Pts. 0, 3, 8 and 10.5 are computed as shown before, positive and negative both being entered on table 9.

Open Spandril Arch Computations

In open spandril arches concentrated live loads and concentrated dead loads computations are needed instead of the uniform live load computations as shewn before.In Col. 76 positive and negative uniform live loads are replaced by positive and negative concentrated dead loads.

In the open spandril type the floor is carried on beams, girders, and columns which rest on arch ribs thus carrying the loads to the ribs as concentrated live and dead loads.

Columns are placed at Pts. 3, 6, 9, 9', 6' and 3' and computations are made as follows as if the arch here designed was carrying concentrated column loads instead of uniform loads in order to show how this is done.

The columns are 20 ft. apart, and the column to the left of Pt. 3 and to the right of Pt. 3' will be off of the arch rib and on the abutments.

The floor is taken as a slab 18 feet wide and $7\frac{1}{2}$ inches thick, carried on floorbeams 10 feet apart and 18" x 18" in size below the floor. These are framed into girders over the columns, these girders being 36" wide x 36" deep. The columns are 12 inches square.

Live Load on Any Column = $20 \times 18 \times 125 \times \frac{1}{2} = 22{,}500$ lbs.

Dead Load on Columns = 80,848 lbs.

Method of Summing Cols. 78 and 82.

All the dead load moments are added algebraically, and the corresponding N s are added also. To this moment sum add the plus concentrated live load and temperature moments to produce maximum positive moment. To the N sum add the corresponding Ns; to produce the thrust that goes with this moment the Ns corresponding to the positive concentrated live load and temperature moments were added to the N sum.

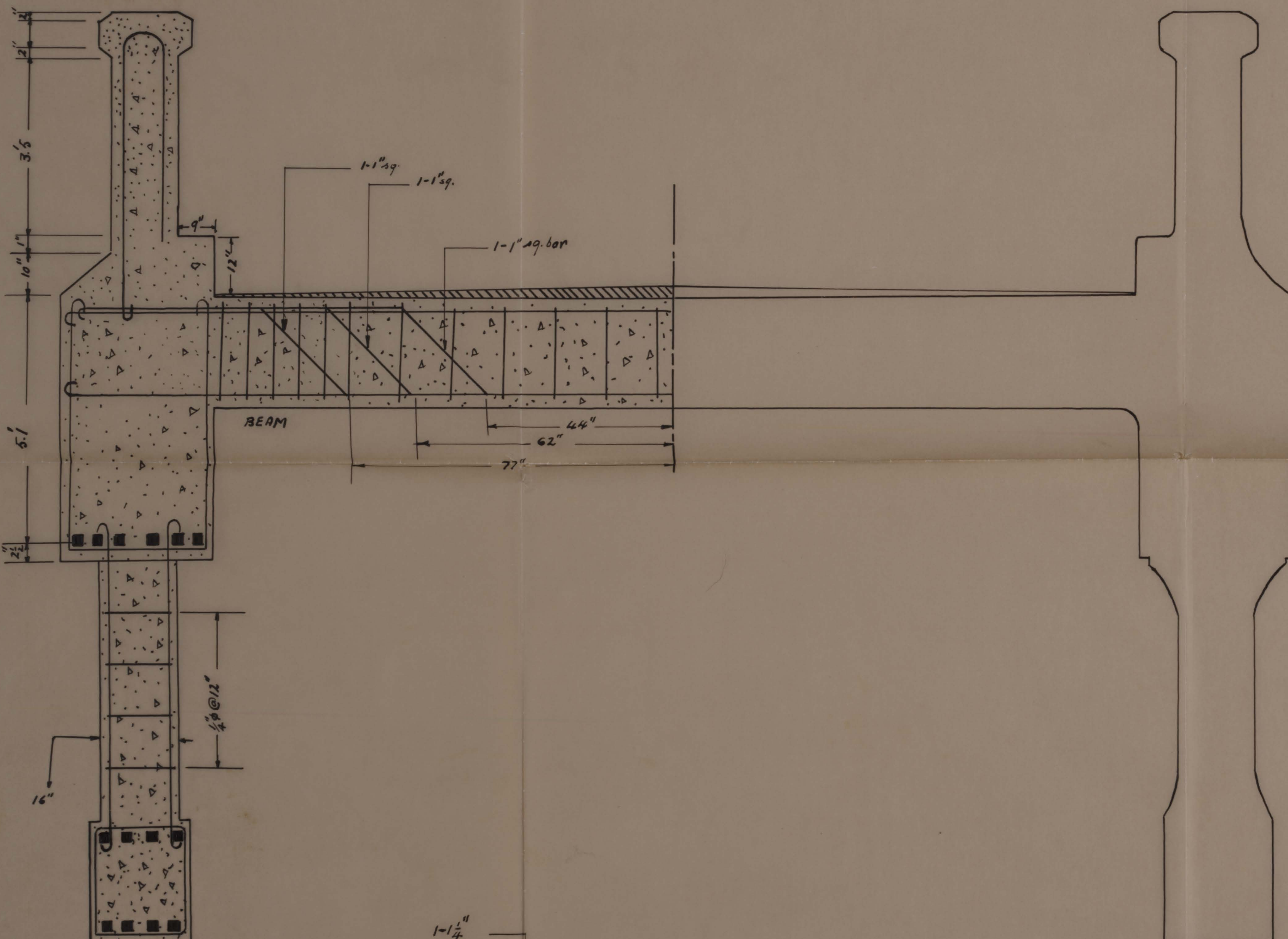
To this same moment sum the minus concentrated live load and temperature moments are added algebraically to obtain the maximum negative moment. To this same sum the corresponding values of concentrated live load and temperature Ns are added to produce the thrust that goes with the minus moment.

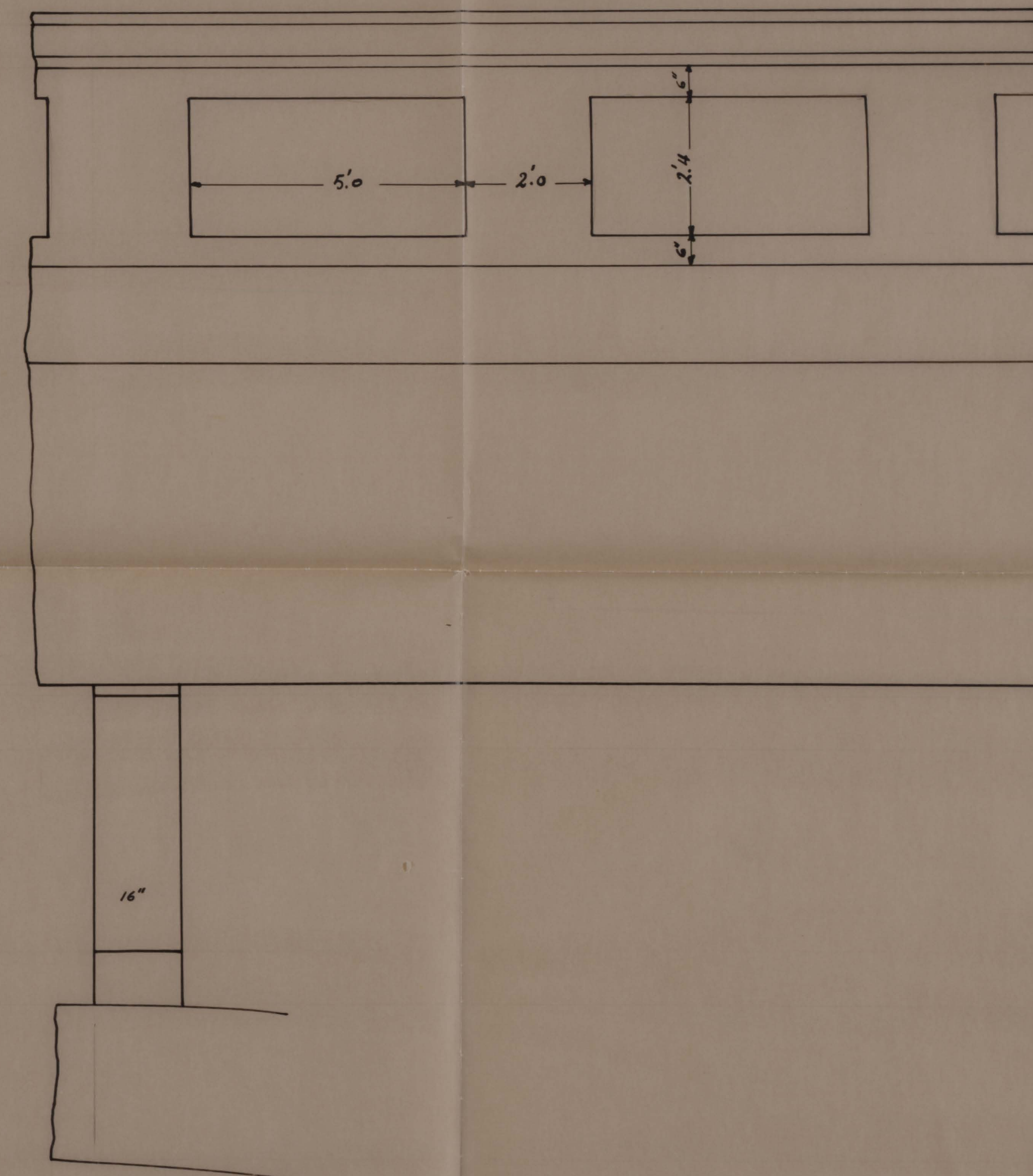
Col. 83 As soon as M and N are known for a point the stress at this point can be computed by diagrams on the basis of the eccentric rectangular column with equal steel in top and bottom.

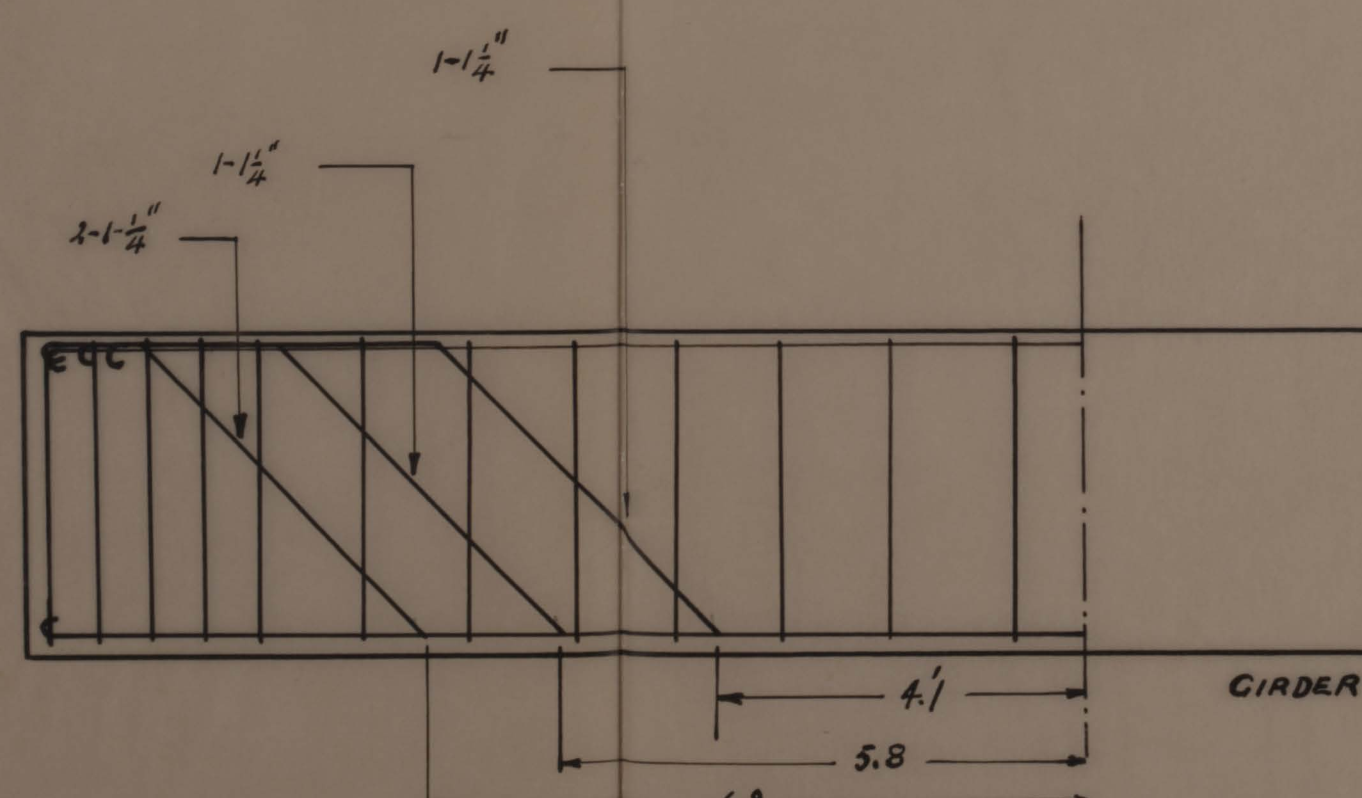
X_0 = eccentricity = M/N p = $A_s/144h$.

If the stresses are greater than the allowable in a filled spandril arch a redesign must be made, but if this occurs in an open spandril arch, since all the loads have been used in an arch rib 12" wide, the arch rib can be widened and new columns 78 and 82 are computed. Dead load and total Ms and Ns will not change but concentrated dead load and concentrated live load Ms and Ns will be reduced and can be found by dividing the values given for a width of 1 foot by the width of the arch rib.

****







A CROSS-SECTION AND A SIDE VIEW OF THE CONCRETE ARCH BRIDGE

GEORGIA SCHOOL OF TECHNOLOGY

DRAWN BY: Yorgi Pavlidis
DATE 1939

Scale: 1" = 2'

Dr. No 2

PART II

DESIGN OF THE THREE-HINGED STEEL ARCH BRIDGE

SPAN = 140 FT.

RISE = 30 FT.

NOTE:

- ○ Shop Rivets
- ● field "
- 7/8 inch. Rivets Used
- 1/2 inch Gusset Pl Used
- 3" Spacing of Rivets

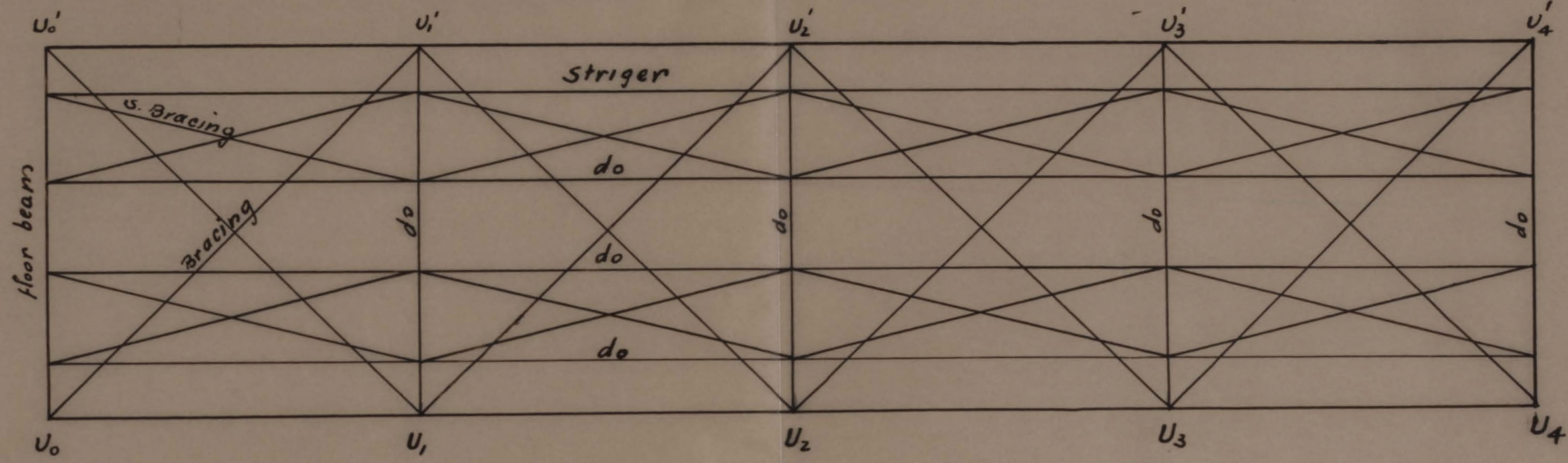
Bottom & Top Chords have double lacing
Diagonales have Single lacing
45° lacing have been used
lacing bars ½"×2"¾

DESIGN OF A RIVETED
THREE-HINGED ARCH HIGHWAY BRI.
SPAN 140'-RISE 30'

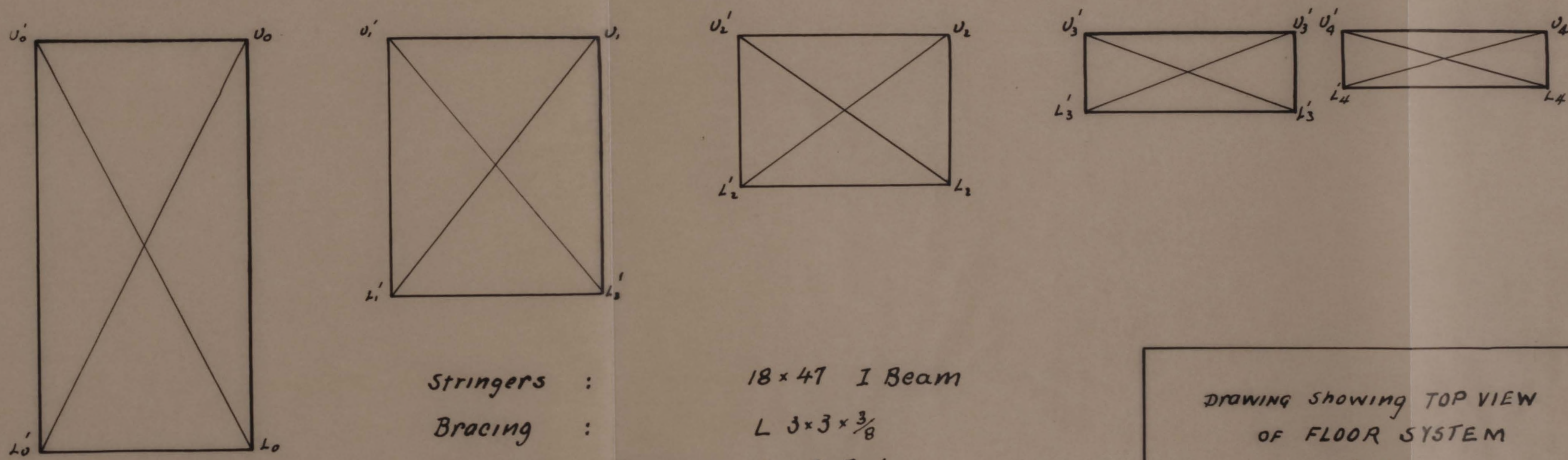
GEORGIA SCHOOL OF TECHNOLOGY

DESIGNED BY: Jorge Pavlidis DRAWN BY:		
DATE 1939	MEMB. SCALE: 1"=10" TRUSS " : 1"=2.5'	Dr. Nº 5

Top view of the floor System



CROSS FRAMES



Stringers	:	18 × 47 I Beam
Bracing	:	L 3 × 3 × 3/8
Cross Frame Bracing	:	2 Ls 8 × 8 × 1/2

DRAWING SHOWING TOP VIEW OF FLOOR SYSTEM AND CROSS FRAMES

Drawn By: Jorgi Pavlidis

DATE 1939

Scale: 1" = 7', Floor System; 1" = 10'

Dr. № 6

DESIGN OF THE THREE-HINGED STEEL ARCH BRIDGE

The three-hinged steel arch bridge was preferred to a simply supported span, because the conditions of the site favor the structure, the arch shape and its proportions, beside that the conditions of the foundation of the site are able to resist a heavy horizontal reaction component. The span of the arch is 140 feet and has a rise of 30 feet which gives a ratio of span over rise equal to 7.

The arch has a parabolic lower chord which results in interesting stress conditions. For a uniform load over the whole structure the diagonals and the top chord carry no stress and the horizontal component of lower chord stress is constant and equal to the horizontal component of reaction thrust.

The three hinged steel arch is a statically determinate structure. The first important principle to keep in mind in dealing with three-hinged arches is as follows: for any single load the line of action of the reactions on the unloaded segment passes through the intermediate hinge, since otherwise there would be a moment at the hinge, which is not possible. The line of actions of the reactions on the loaded side passes through the intersection of the lines of action of the load and the other reaction since the lines of action of three non-paralell forces acting on a rigid body in equilibrium meet at a point. The locus of all such load-reaction-lines intersections is called the position line, which is often used in an analysis .

The following articles contain the complete design and the detai of the riveted three-hinge arch, designed to carry two lanes of normally heavy traffic. The design is based upon the Standard Specifications on Highway Bridges of the American Association of Highway Officials and the American Railway Engineering Association, and the Specifications for Steel Highway Bridges of the American Society of Civil Engineers.

Here below we give the general data which is required in order the design can be made.

Span Center to Center of Bearings = 140 ft.

Number of Panels = 9

Width of Roadway = 18 ft.

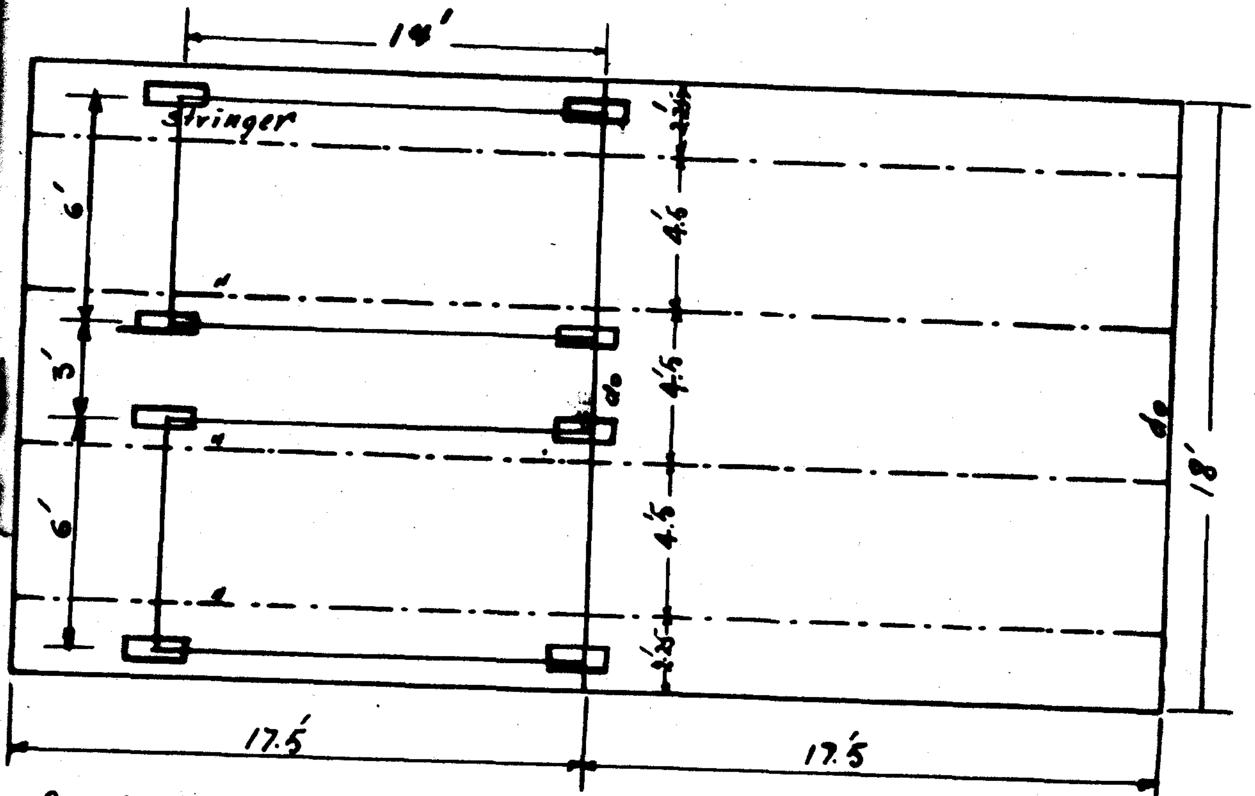
Rise of the Arch = 30 ft.

Live Load = Class H-15

All rivets are 7/8 in. except they are noted otherwise.

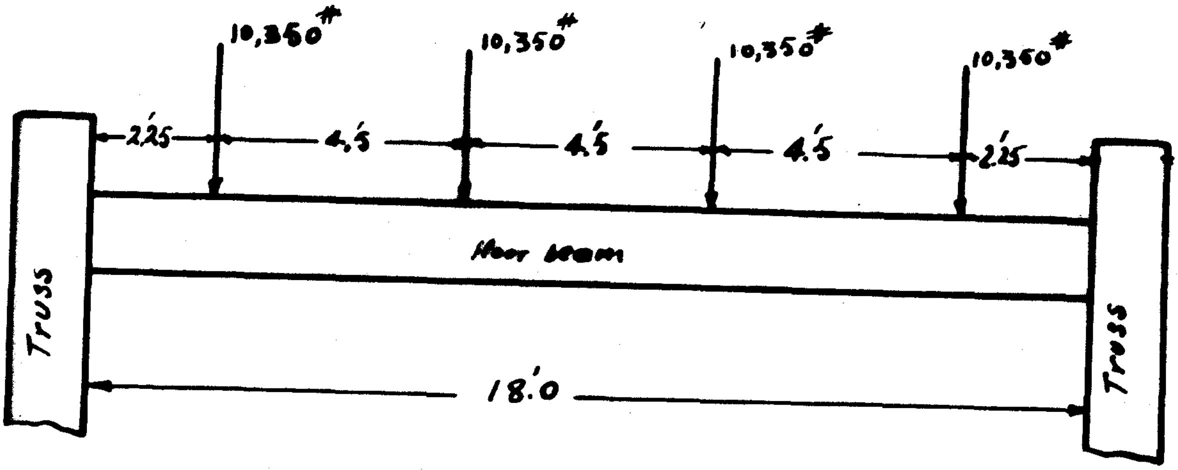
FLOOR SLAB

The floor is to be 2" of bituminous macadam on a reinforced concrete $7\frac{1}{2}$" slab, supported on longitudinal stringers. Four stringers will be used. The centers of the outside stringers may be placed some little distance inside of the clearance line on account of the width of the flange and increased strength of the slab at its junction with the curb. The distance center to center of the outside stringers is 13ft. 6in. The concrete slab is continuous over the stringers. Fig. A shows the position of the stringers and floor beams with their respective distances.



14'
Stringer
6'
3'
6'
2.25'
4.5'
4.5'
4.5'
2.25'
do
do
18'
17.5'
17.5'

fig. A, C



10,350#
10,350#
10,350#
10,350#
2.25'
4.5'
4.5'
4.5'
2.25'
floor beam
Truss
Truss
18.0'

fig. B, D

STRINGER DESIGN

Computations:

Span of Stringers Center to Center of Floor Beam = 17.5 ft.

Dead Load on the End Stringers

$4.5 \times 120 = 540$ lbs./ft.

Assume Weight of Stringer = 50 lbs. / foot

Total Dead Load

$540 + 50 = 590$ lbs./ft.

Maximum Concentrated Live Load at Center of Span of Each Interior Stringer = 12,000 lbs.

Coefficient of Impact

$50/(1 + 125) = .350$

Moment

Dead Load Moment

$1/8 \times 500 \times (17.5)^2 \times 12$ = 270,000 "lbs.

Live Load Moment

$1/4 \times 12{,}000 \times 17.5 \times 12$ = 630,000 "lbs.

Impact Moment

$.350 \times 630{,}000$ = 220,000 "lbs.

Total Maximum Moment = 1,120,000 "lbs.

Section Modulus Required

$$S = M/fs = \frac{1{,}120{,}000}{16{,}000} = 70 \text{ in.}^3$$

Try 16" wide Flange I Beam

Weight = 45 lbs./ft.

Section Modulus = 72.4

$b = 7.039$

$l = 17.5$

$$l/b = \frac{17.5 \times 12}{7.039} = 29.9$$

$f_s = 13{,}850$

$$S = \frac{1{,}120{,}000}{13{,}850} = 81 \text{ in}^3$$

Try 18" Wide Flanged I Beam

Weight $= 47$ lbs./ft.

Section Modulus $= 82.3 \text{ in}^3$

$b = 7.492$

$l = 17.5$

$$l/b = \frac{17.5 \times 12}{7.492} = 28.0$$

$f_s = 14{,}300$

$$S = \frac{1{,}120{,}000}{14{,}370} = 78 \text{ in.}^3$$ (less than 82.3) All right.

Use 18" x 47 lbs./ft. I Beam.

FLOOR BEAM DESIGN

Computations:

Dead Load from the Stringers

$590 \times 17.5 = 10,350$ lbs.

Moment

Dead Load Bending Moment Due to these Concentrations

$(20,700 \times 6.75 - 10,350 \times 4.5)\ 12 = 1,120,000$ "lbs.

Assume Weight of Floor Beam = 100 lbs./ft.

Dead Load Bending Moment Due to this Weight

$1/8 \times 100 \times (18)^2 \times 12 = 48,500$ "lbs.

Total Dead Load Moment

$1,120,000 + 48,500 = 1,168,500$ "lbs.

The maximum live load moment occurs when the loads are placed as indicated in Fig. C. The actual live load stringer reactions with the loads as placed are as follows:

Interior Stringer Load

$\left(12,000 + \frac{3000 \times 3.5}{17.5} \right) = 12,600$ lbs.

Outside Stringer Load

$\left(12,000 + \frac{3000 \times 3.5}{17.5} \right) = 12,600$ lbs.

The position of the live loads are shown in Fig. D.

Live Load Moment

$(25,200 \times 6.75 - 12,600 \times 4.5)\ 12 = 1,360,000$ "lbs.

Impact Moment

$\left(\frac{50}{36 + 125} \right. = 540,000$ "lbs.

Total Moment

$1,168,500 + 1,360,000 + 540,000 = 3,068,500$ "lbs.

Shear

Shear of Live Load	=	25,200 lbs.
Shear of Impact	=	7,800 lbs.
Shear of Dead Load	=	20,700 lbs.
Total Shear	=	63,700 lbs.

Section Modulus Required

$$\frac{3,068,500}{16,000} = 192 \text{ in.}^3$$

Try 24" Wide Flanged I Beam

Weight = 87 lbs./ft.

Section Modulus = 204.3

$A = 25.58 \text{ in.}^2$

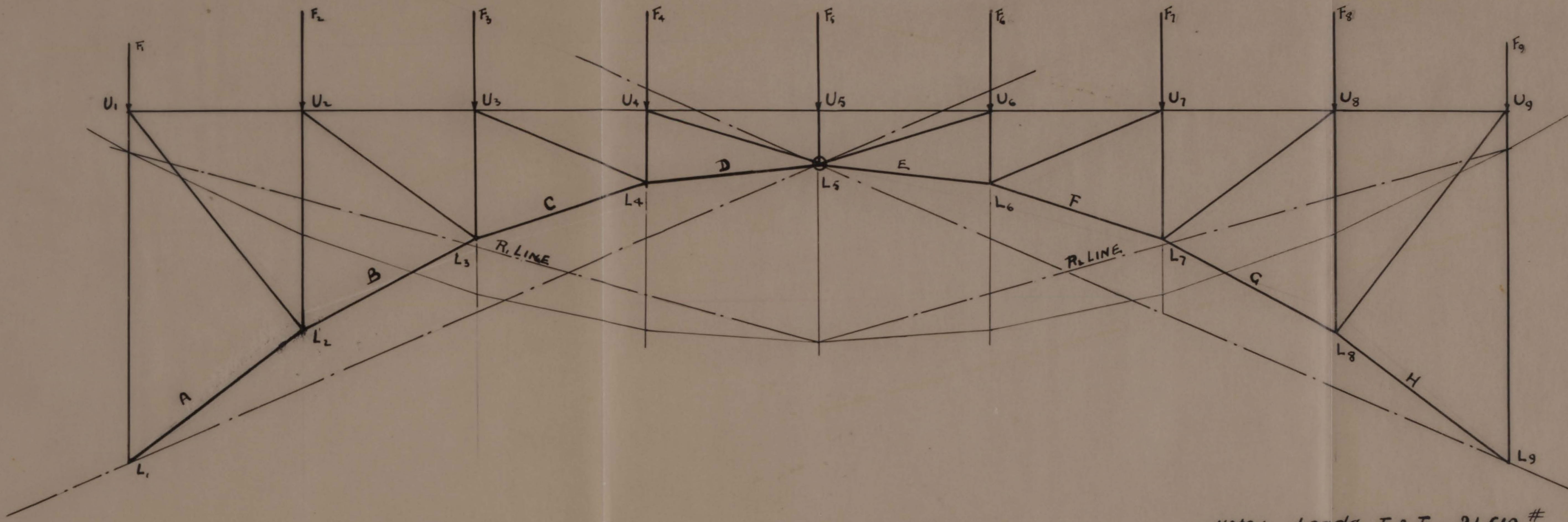
$b = 9.025$

$l = 18$

$$l/b = \frac{18 \times 12}{9.025} = 24$$

$$S = \frac{3,068,500}{15,530} = 198 \text{ (less than 204.3) All right.}$$

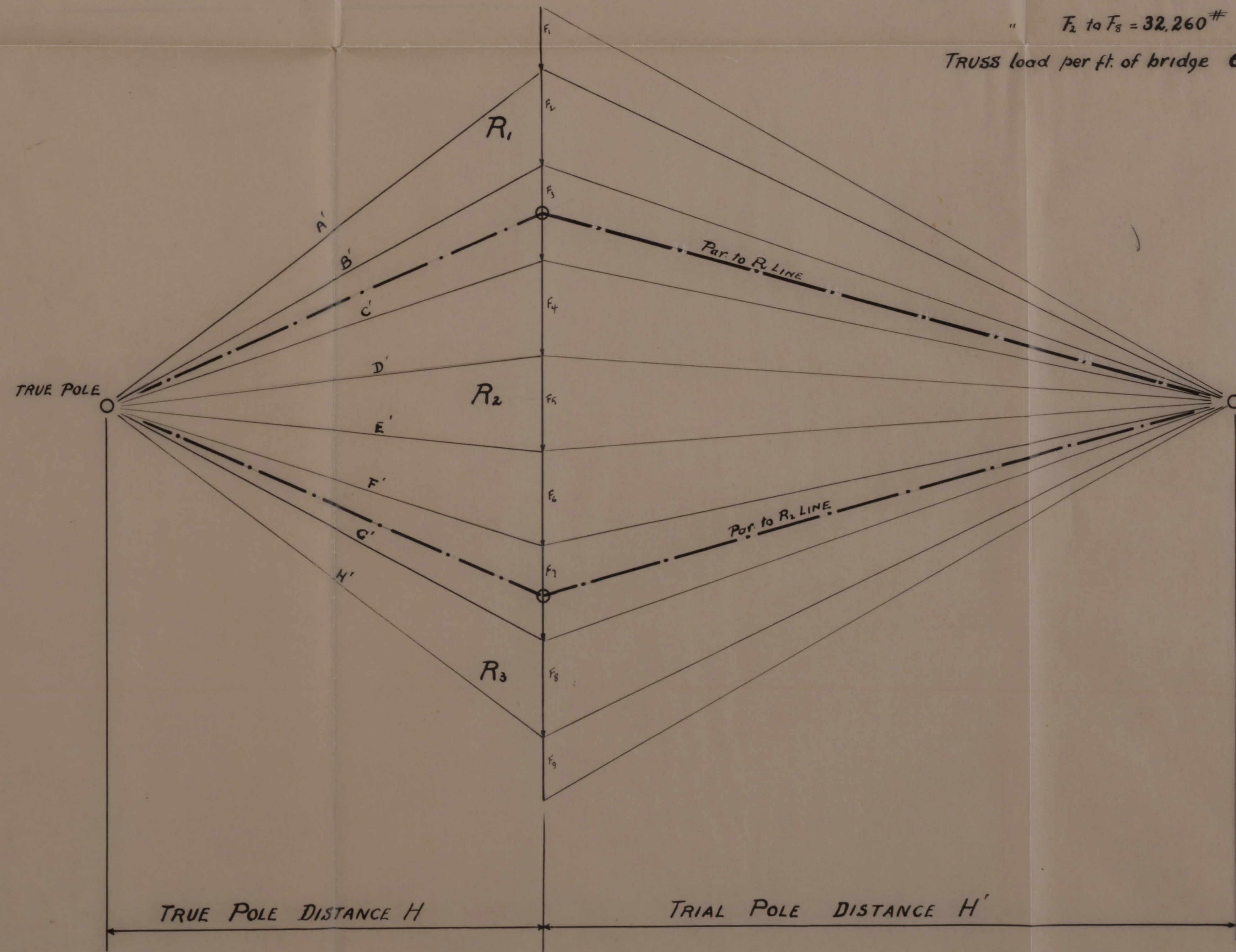
Use 24" x 87lbs./ft. I Beam.



Note: Loads F_1 & F_9 = 21,510 #

" F_2 to F_8 = 32,260 #

TRUSS load per ft. of bridge 634 #



GA. SCHOOL OF TECH.

DIAGRAM

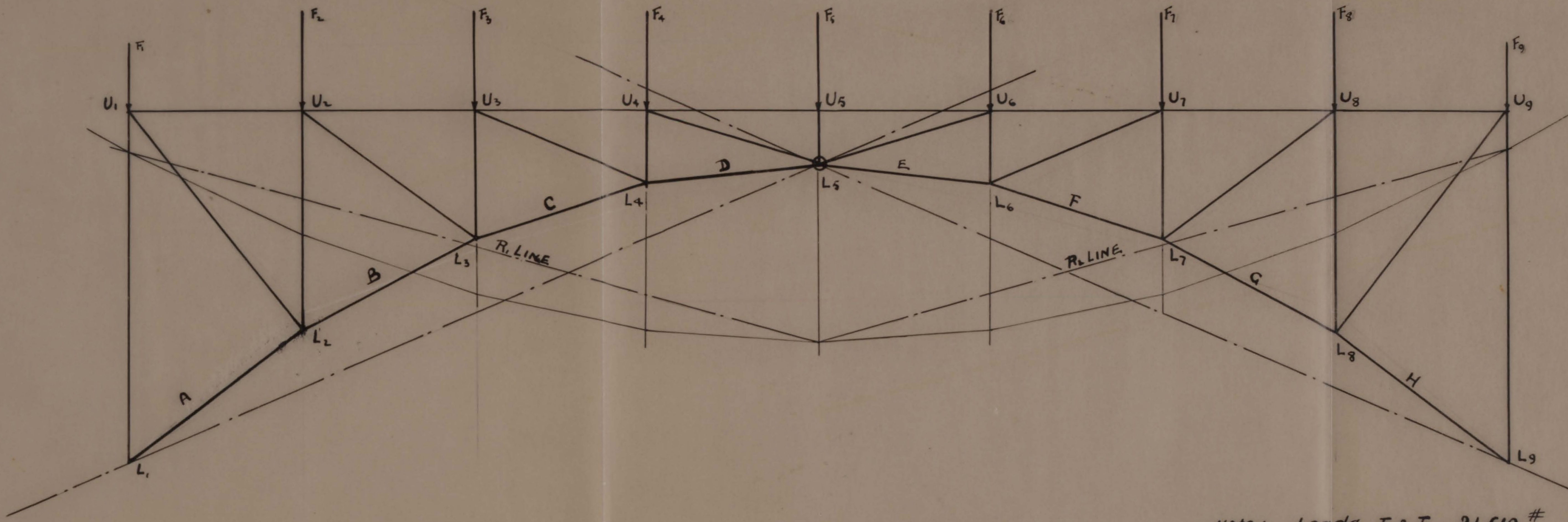
FOR

D.L. STRESSES 3-HINGE STEEL ARCH

YEAR 1939 Y. PAVLIDIS

Scale for truss 1" = 10' Diag. 1" = 32,260 #

Drawing No 1



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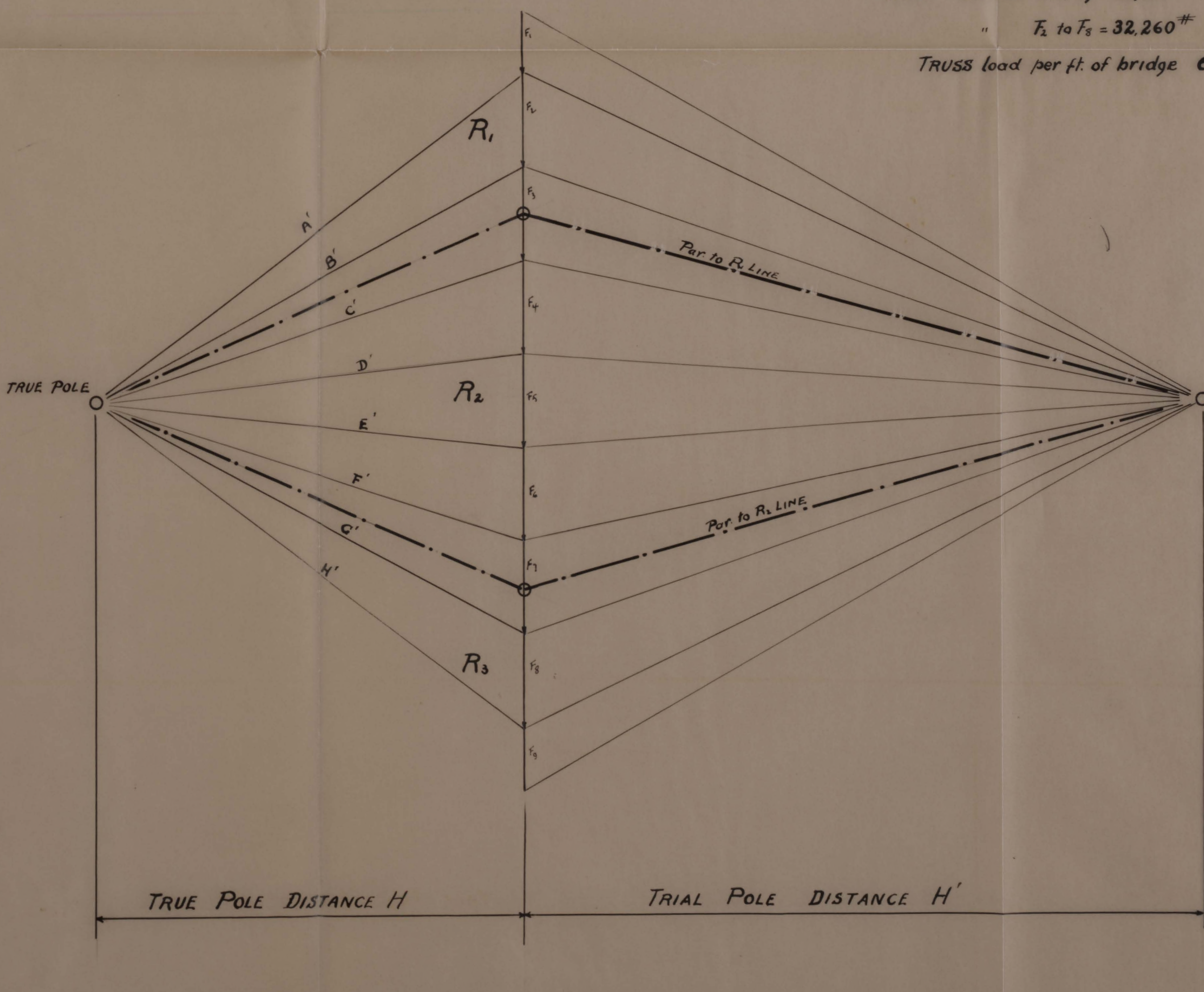


GA. SCHOOL OF TECH.

DIAGRAM

FOR

D.L. STRESSES 3-HINGE

STEEL ARCH

YEAR 1939 Y. PAVLIDIS

Scale for truss 1" = 10′ Diag. 1" = 32,260 #

Drawing No 1

DEAD LOAD OF TRUSS

The dead load weight of the trusses and bracings may be approximated by the equations of Urquhqrt and O'Rourke, namely:

$$w = 21 + \frac{l(w_1 - 1000)}{2000}$$

where w = weight per foot of trusses and bracing.

l = length of the span.

w_1 = Total floor load (dead, live, or impact) in pounds per foot of span.

Computations

Dead Load of Floor system of ½ of the Bridge

Dead Load of Slab & Stringer	= 20,700 lbs.
Dead Load of Floor Beam	= 810 lbs.
Total Dead Load	= 21,510 lbs.
Live Load	= 25,000 lbs.
Impact	= 8,100 lbs.
Total Dead load + Live Load + Impact	= 54,610 lbs.

Weight of Truss + Bracings per ft. of Truss

$$w = 2(140) + \frac{140\ (6060 - 1000)}{2000} = 634 \text{ lbs./ft.}$$

Dead Load at Lower Panel Points

634 x 17 = 10,750 lbs.

Dead Load at Upper Panel Points = 21,510 lbs.

The dead load of the stresses of the members of the truss are found graphically by the use of the Maxwell diagram as shown in the drawing Number I. Theresults obtained are listed below in order.

$U_1L_1 =$	- 21,510 lbs.	$L_1L_2 =$	- 189,000 lbs.
$U_2L_2 =$	- 32,200 lbs.	$L_2L_3 =$	- 172,000 lbs.
$U_3L_3 =$	- 32,200 lbs.	$L_3L_4 =$	- 159,000 lbs.
$U_4L_4 =$	- 32,200 lbs.	$L_4L_5 =$	- 152,200 lbs.
$U_5L_5 =$	- 32,200 lbs.		

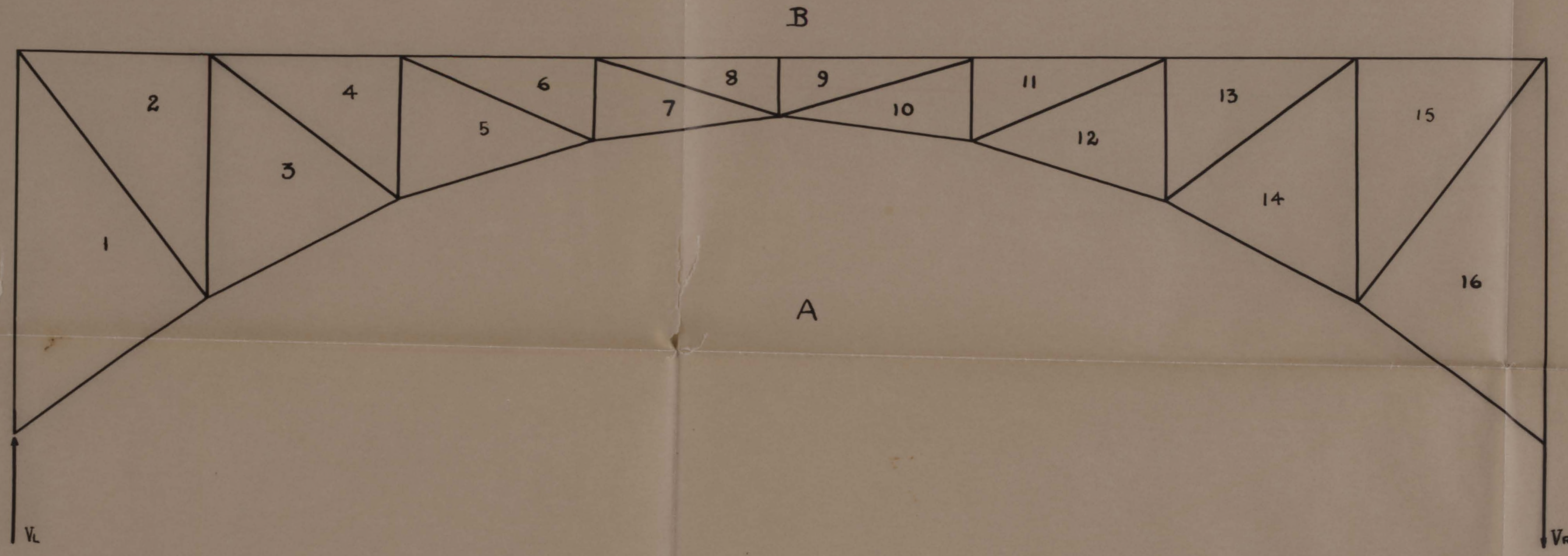
LIVE LOAD STRESSES

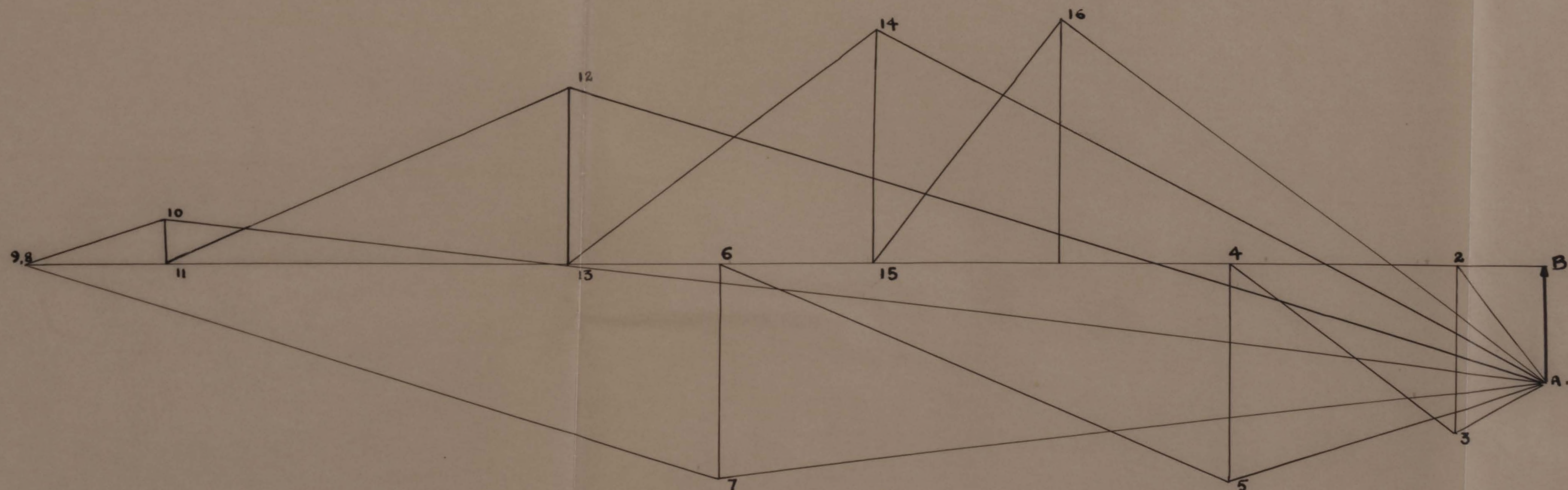
The live load stresses are most conveniently determined by means of the influence lines, as these show in what manner the load has to be placed in order to produce maximum stresses. For the determination of the stresses the following influence lines have been drawn.

Influence Lines for Truss Members.

The stress in any member of the arch truss due to a certain, loading, the arch being considered as a single span, and the stress being caused by the sole action of the horizontal reaction H. The influence area for that member is therefore, the geometric sum of the influence area for simple span and the influence area due to H. The influence area due to H is a triangle with apex C, since the stress caused in the member by H varies in linear proportion with H.

In finding the stress in member L_1L_2 for instance, triangle ABC represents the positive influence area for L1L2, the truss being considered a single span AC and a triangle AB'C which is the negative influence area due to H. The resulting influence area is shown as negative in triangle KB'C and as positive in triangle ABK. A load P equal to 1 (one) at B' causes a horizontal reaction of H = ab/lf and the ordinate at B' of the triangle AB'C is equal to the stress caused in member L_1L_2 by the reaction.





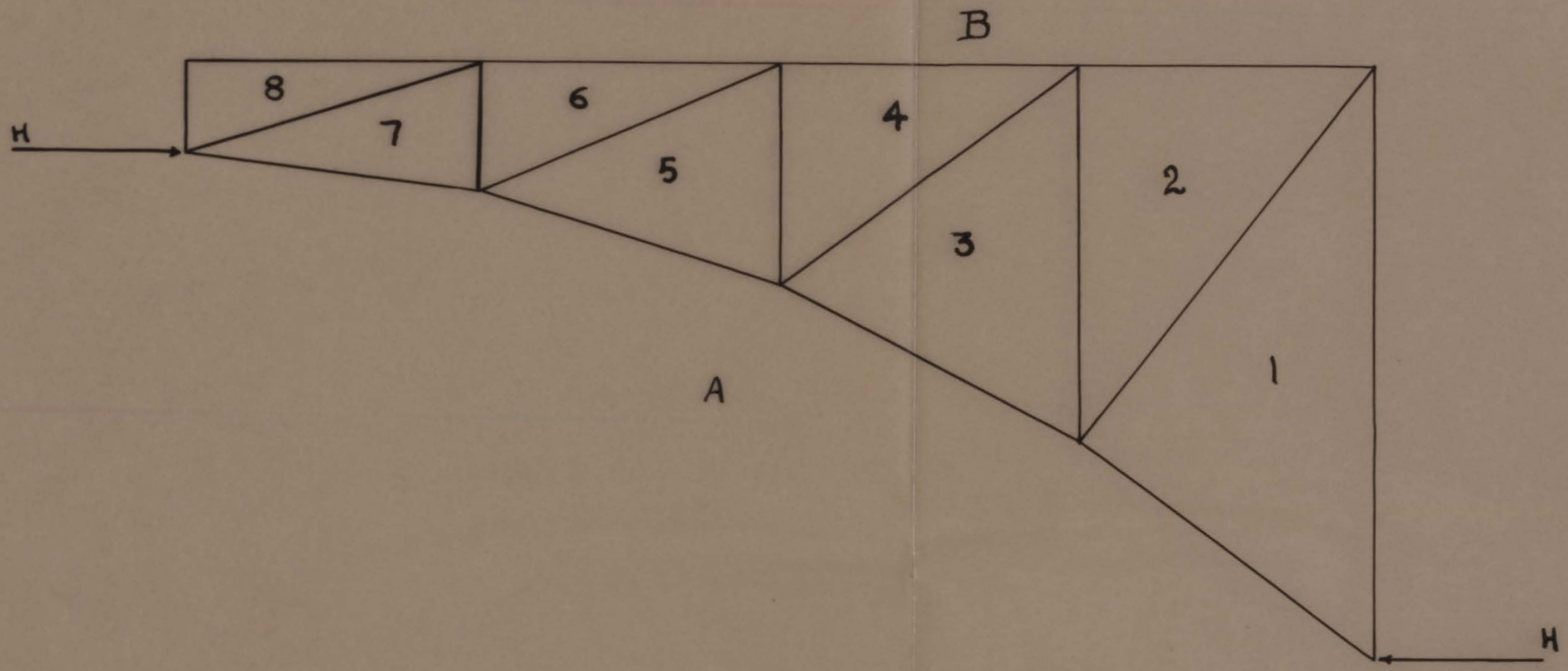
C.A. SCHOOL OF TECHNOLOGY

DIAGRAM FOR ORDINATES IN INFLUANCE LINES
DUE TO A VERTICAL FORCE $V_L = 1^{\#}$

YEAR 1939 Y. PAVLIDIS.

Scale for truss 1"=10' Diag. 1"=1.#

Drawing Nº 3



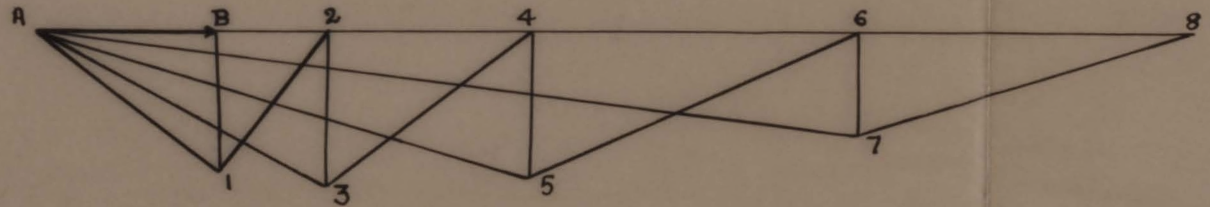


DIAGRAM FOR ORDINATES IN
INFLUEANCE LINES
DUE TO A HORIZONTAL FORCE H = 1.16#

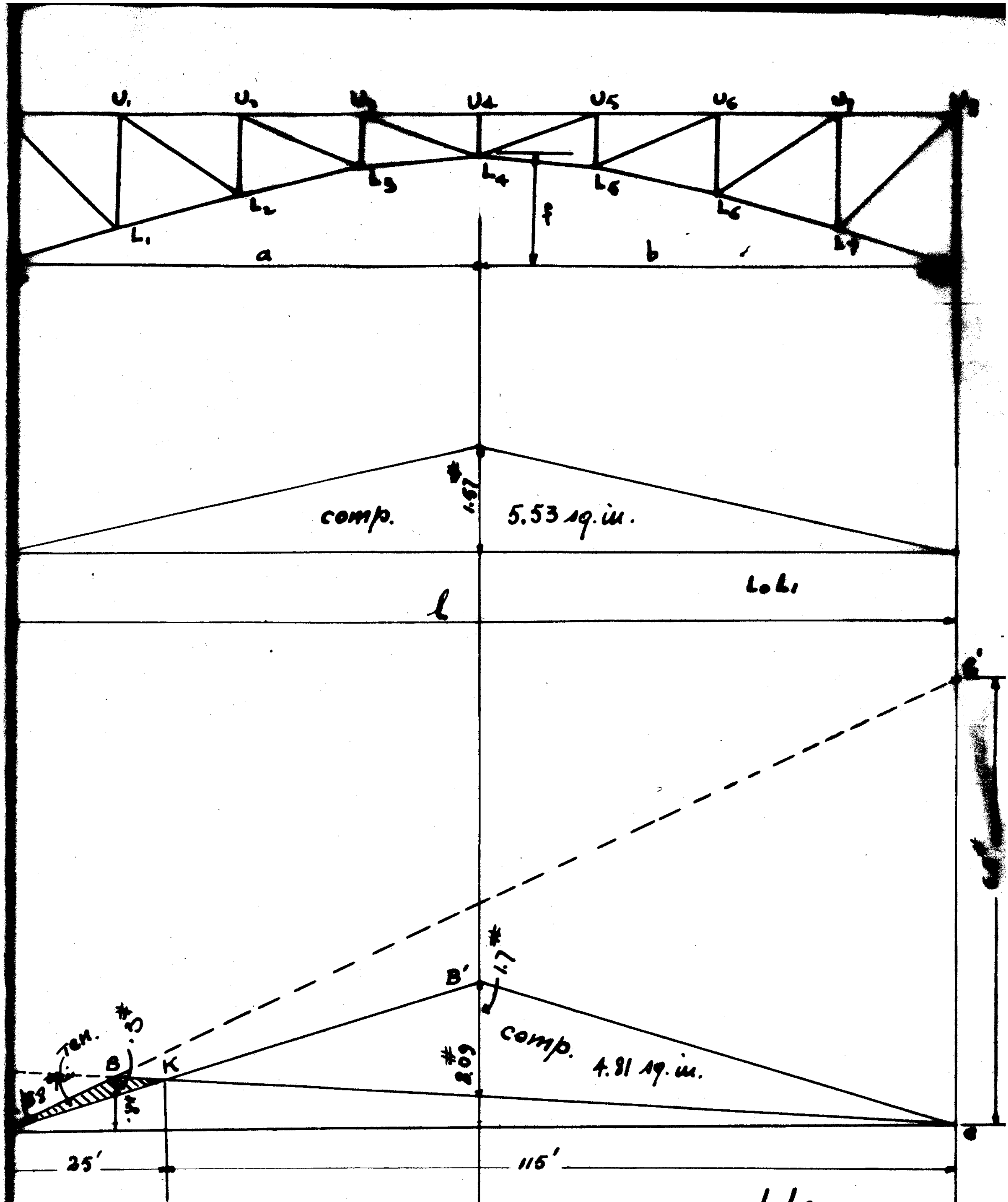
Scale for truss 1" = 10'
Diag: 1" = 1.16#

Drawing № 2

The influence area ABC is found as for simple spans except that the ordinate AA' is not made equal to the load unity, but equal to the stress caused in the member L_1L_2 by an upward load unity at A, the truss being assumed fixed at C, since the ordinates of triangle AB'C also represent the correct or unreduced stress due to H, in other words, the influence ordinates are shown to the correct size and used not be multiplied by an influence coefficient. In a similar way, the influence area for the verticals and diagonals have been used. As a rule the following method has been used: 1- Determinetion of stresses S_L in all members due to a horizontal force H = ab/lf. This is most conveniently done by a maxwell diagram Number 2. Those stresses give the heights of the triangles AB'C.

2- Determination of stresses S_a and S_c in all members due to a vertical upward force $V_a = 1$ at A, the truss being assumed fixed at C. This is best done by another Maxwell diagram Number 3 starting the resolution of stresses at A and ending at C. These stresses are the ordinates AA' anc CC' locating lines CA' and AC', in other words, locating the point B of triangle ABC. Any convenient scale may be used for plotting influence lines. The vertical of the following influence lines is taken as 1" = 2 lb. and the horizontal scale as 1" = 20'.

To determine the concentration or the value of the stress due to a concentrated load, it is neccessary only to multiply the load in pounds by the value of the ordinate in pounds over which the load is placed. For a uniform load per foot of truss the stress in the member can be determined by summing up the values found after multiplying the ordinates for each foot across the bridge by the load per foot. Fortumately however, the same result is obtained by multiplying the area of the diagram by the load per foot of truss.



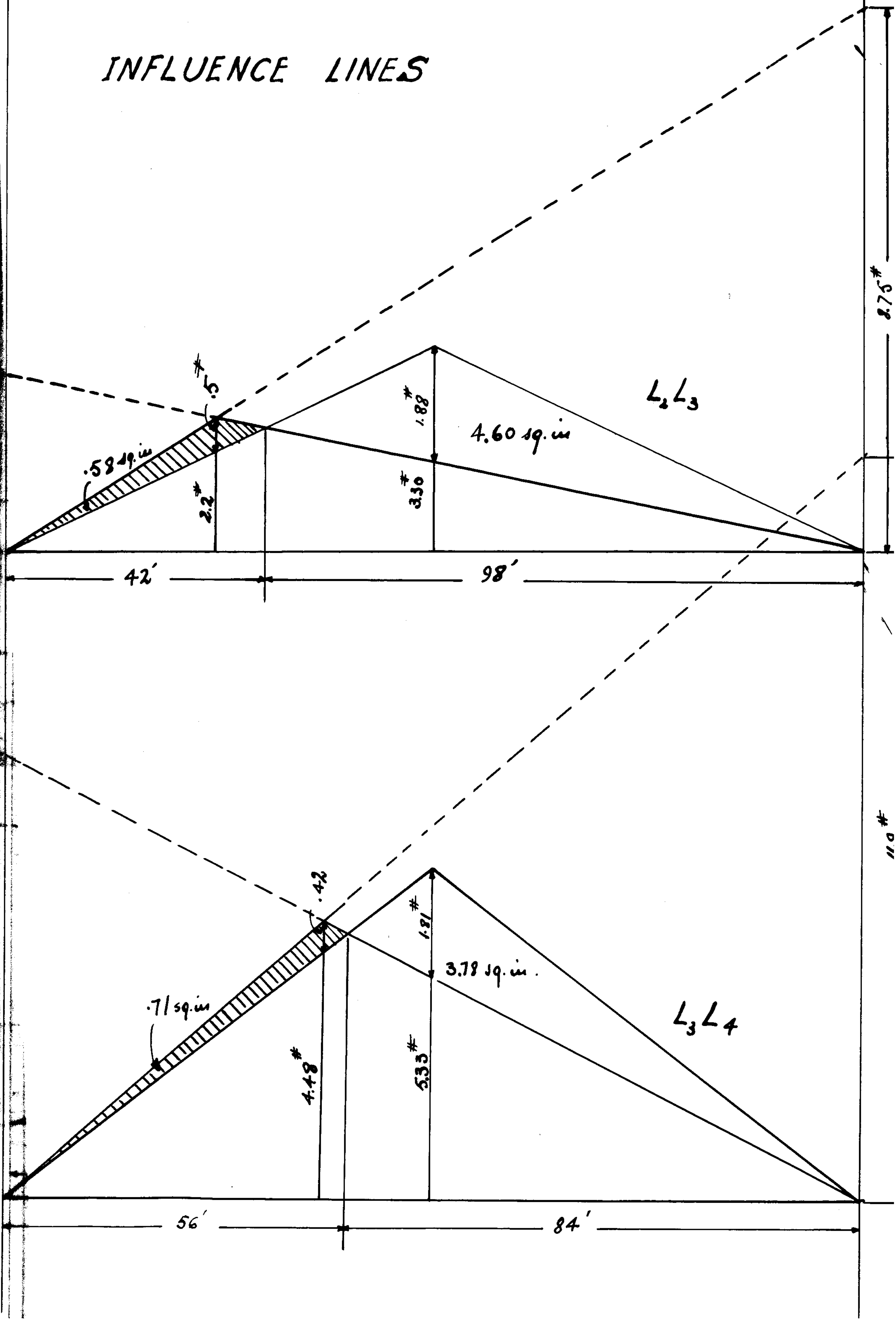
FOR INFLUENCE LINES SHOWN IN THIS THESIS IN CONNECTION WITH 3-HINGED ARCH; ordinates ended by arrows are values which must be multiplied by the concentrated load to obtain stress in lbs.

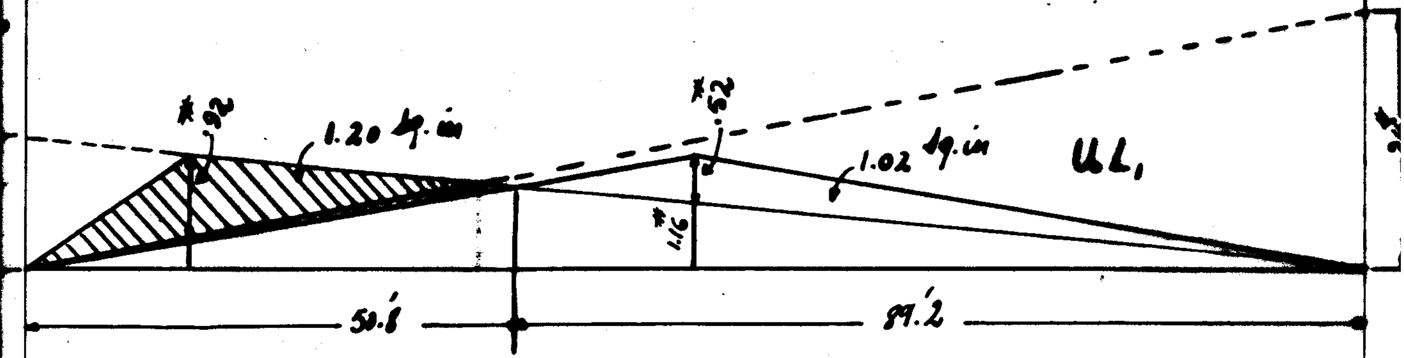
Shaded areas represent tension. Vert. Scale 1" = 2#

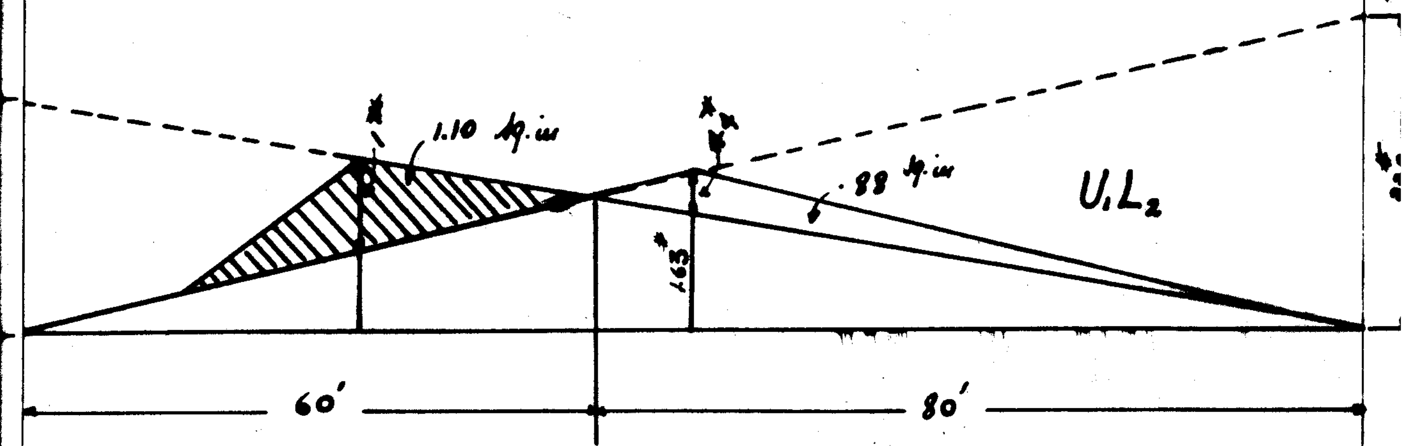
K = CONSTANT TO TRANSFORM AREAS IN lbs. Horiz. " 1" = 20'

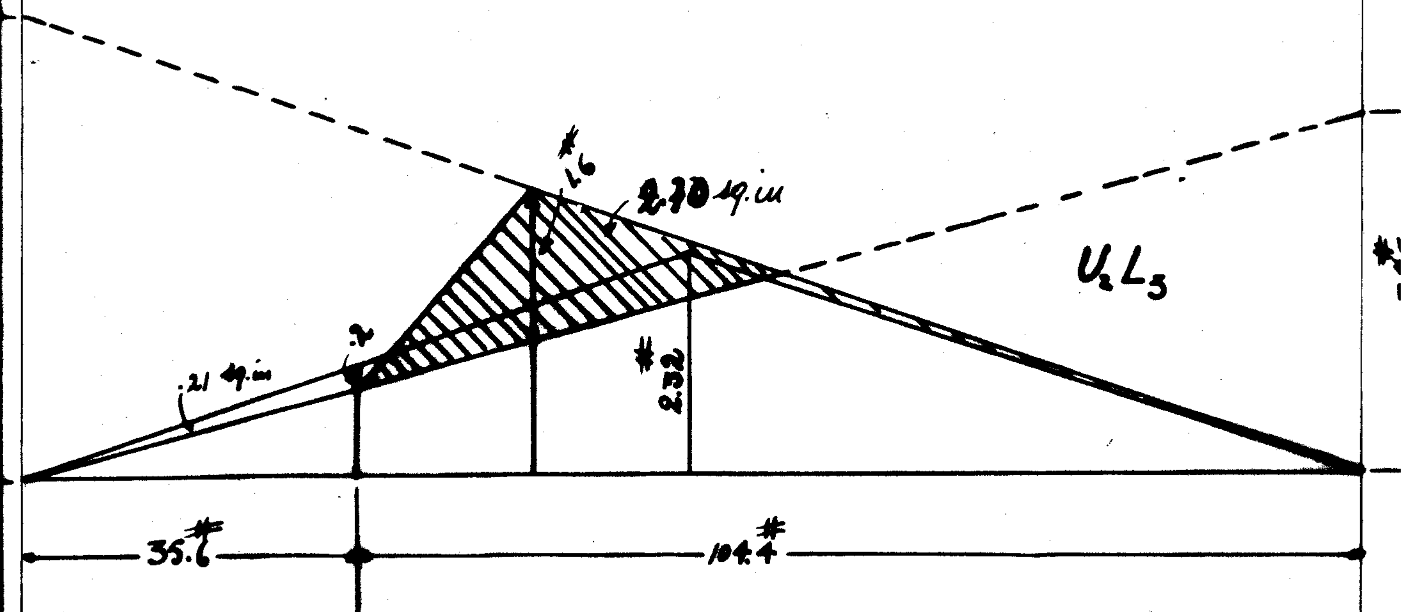
K = 40

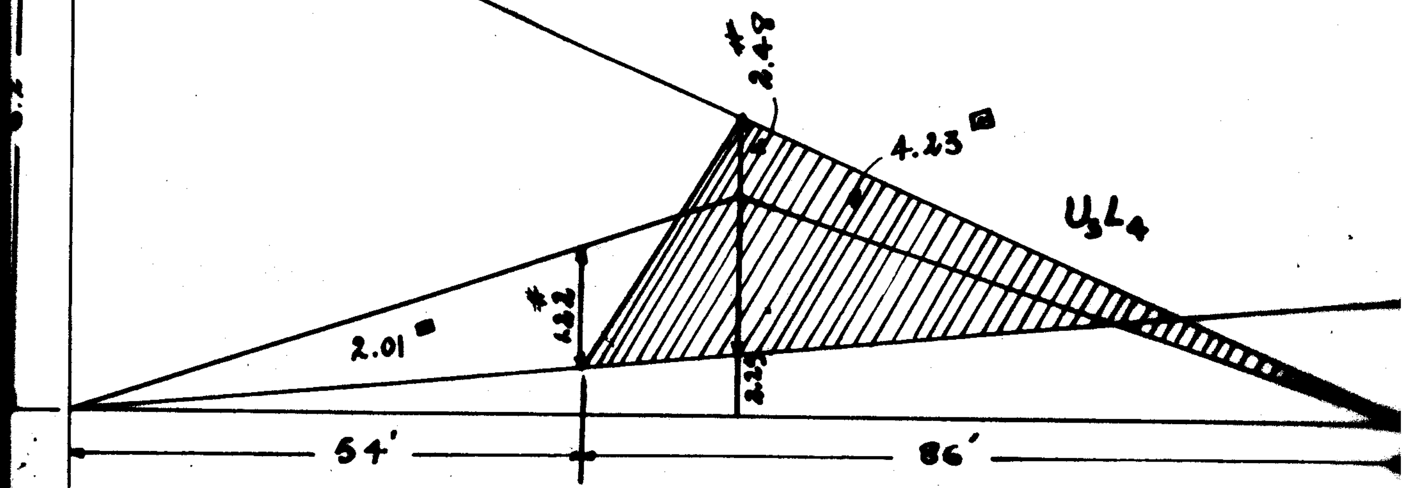
INFLUENCE LINES

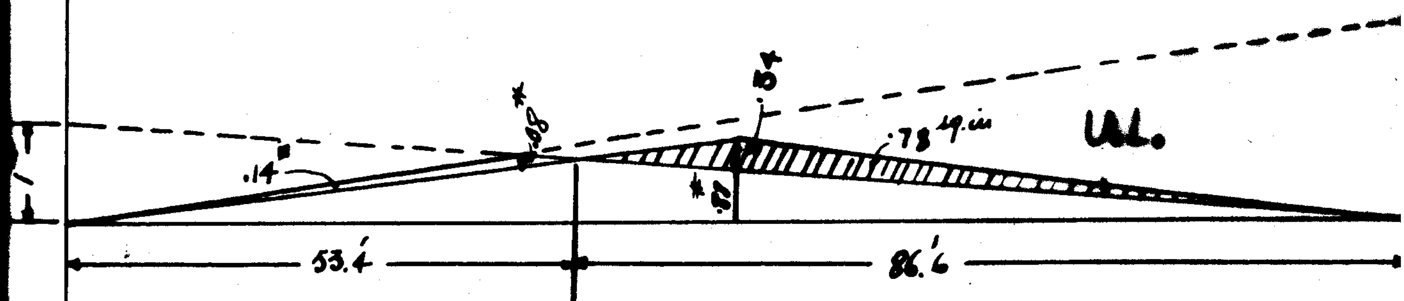


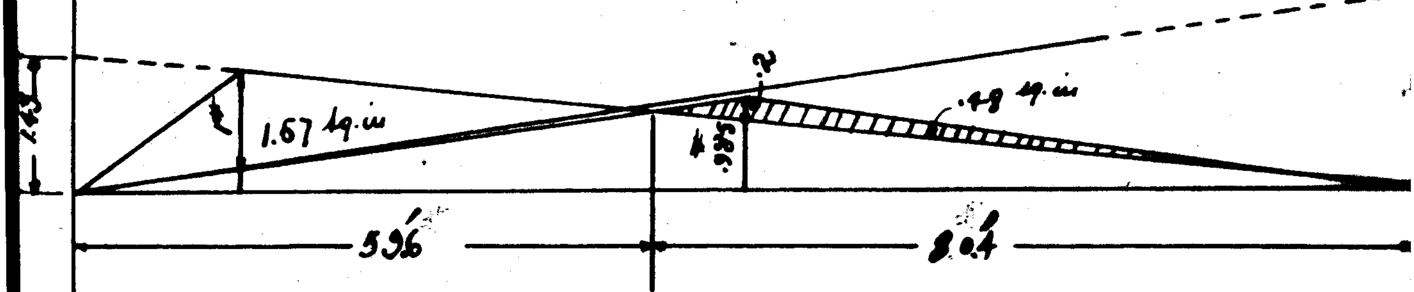

1.20 sq.in
1.02 sq.in
UL1
1.16
50.8'
89.2'

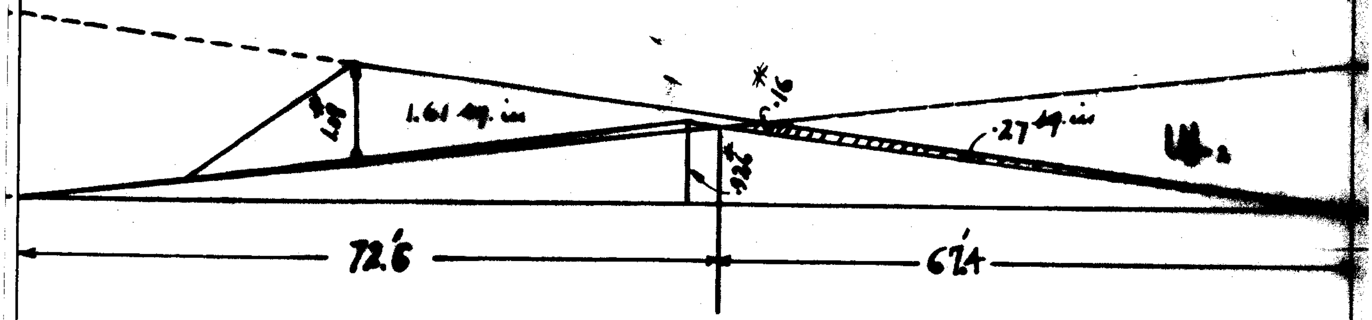

1.10 sq.in
.88 sq.in
U1L2
1.63
60'
80'


.21 sq.in
U2L3
2.32
35.6#
104.4#

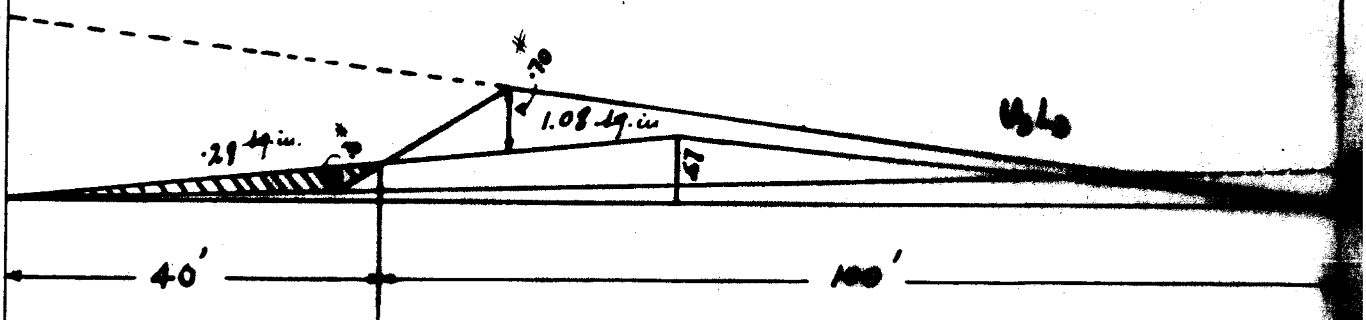

8.48#
4.23□
U3L4
2.01□
4.82#
2.25
54'
86'


.54
.98#
.78 sq.in
U1L0
.14□
#.87
1
53.4'
86.6'

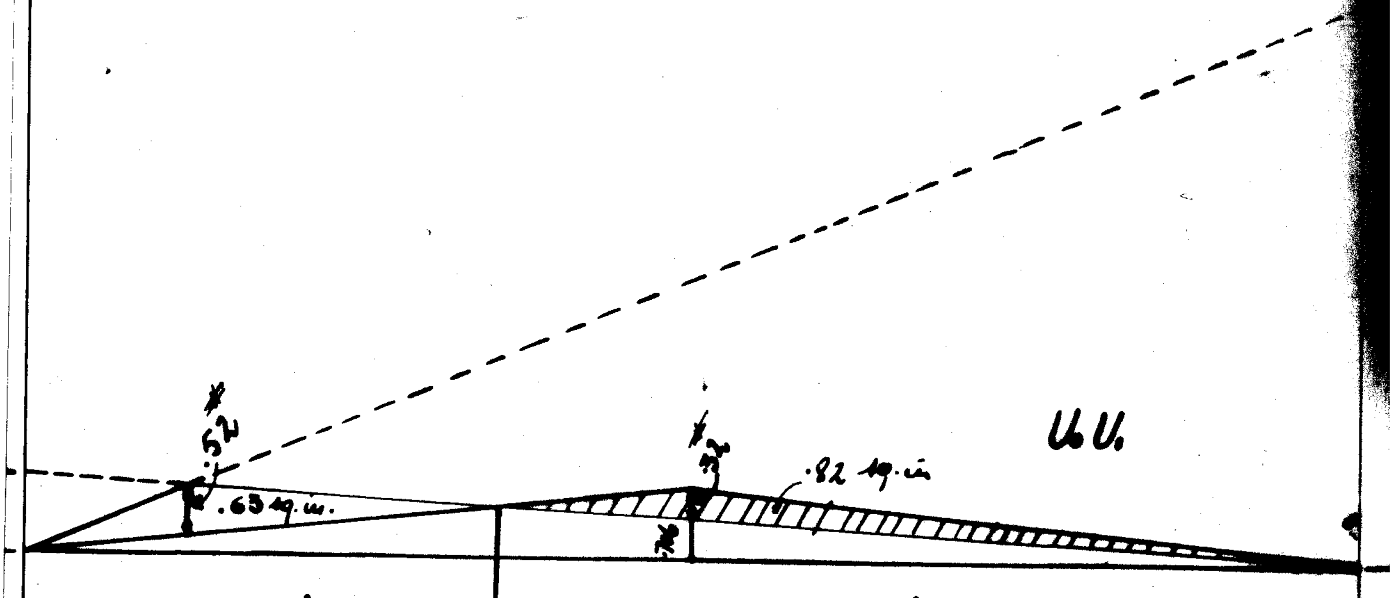

U1L1
1.43
1.67 sq.in
.2
.48 sq.in
#.985
59.6'
80.4'



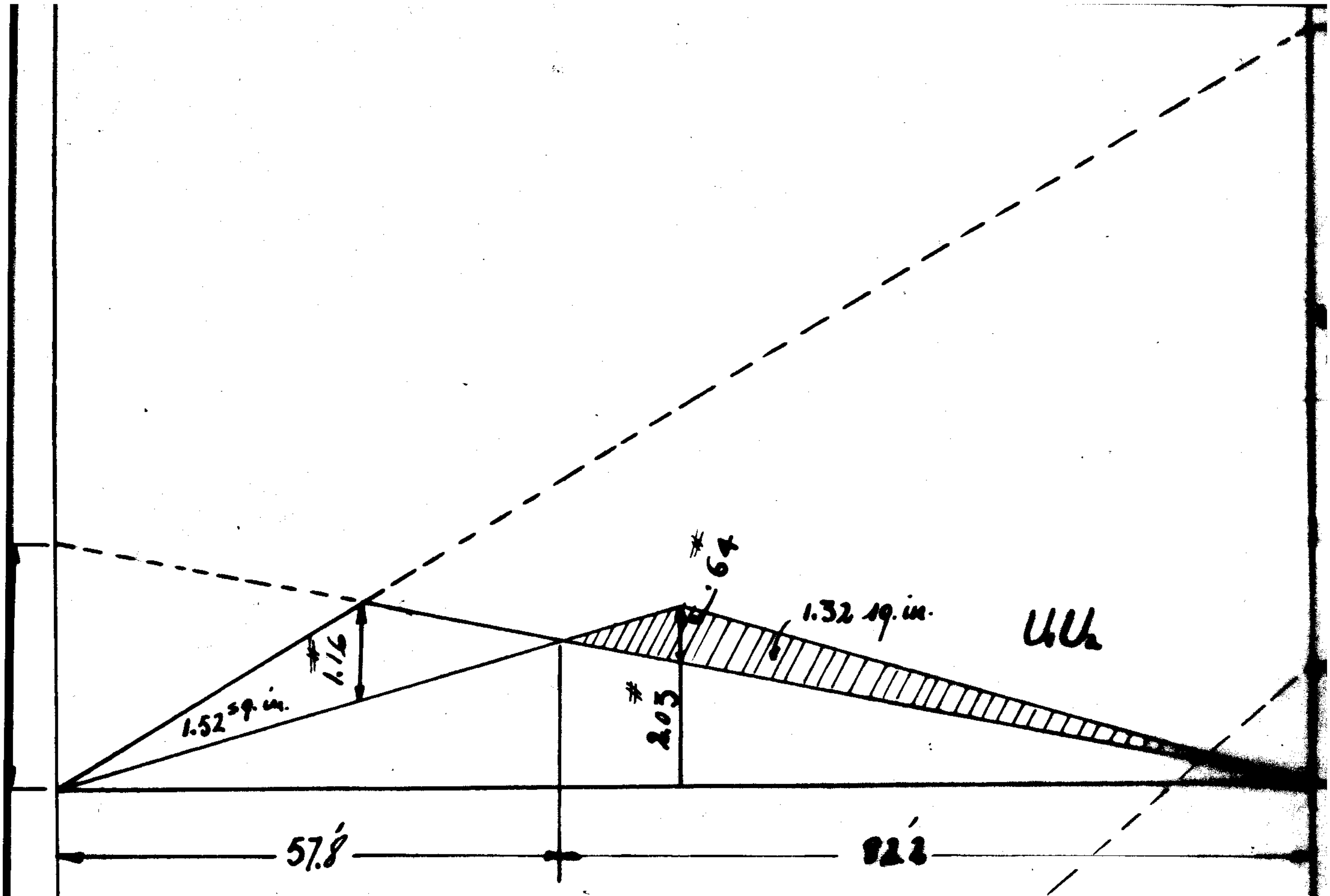
1.61 sq. in
.16
.27 sq. in
72.6
62.4



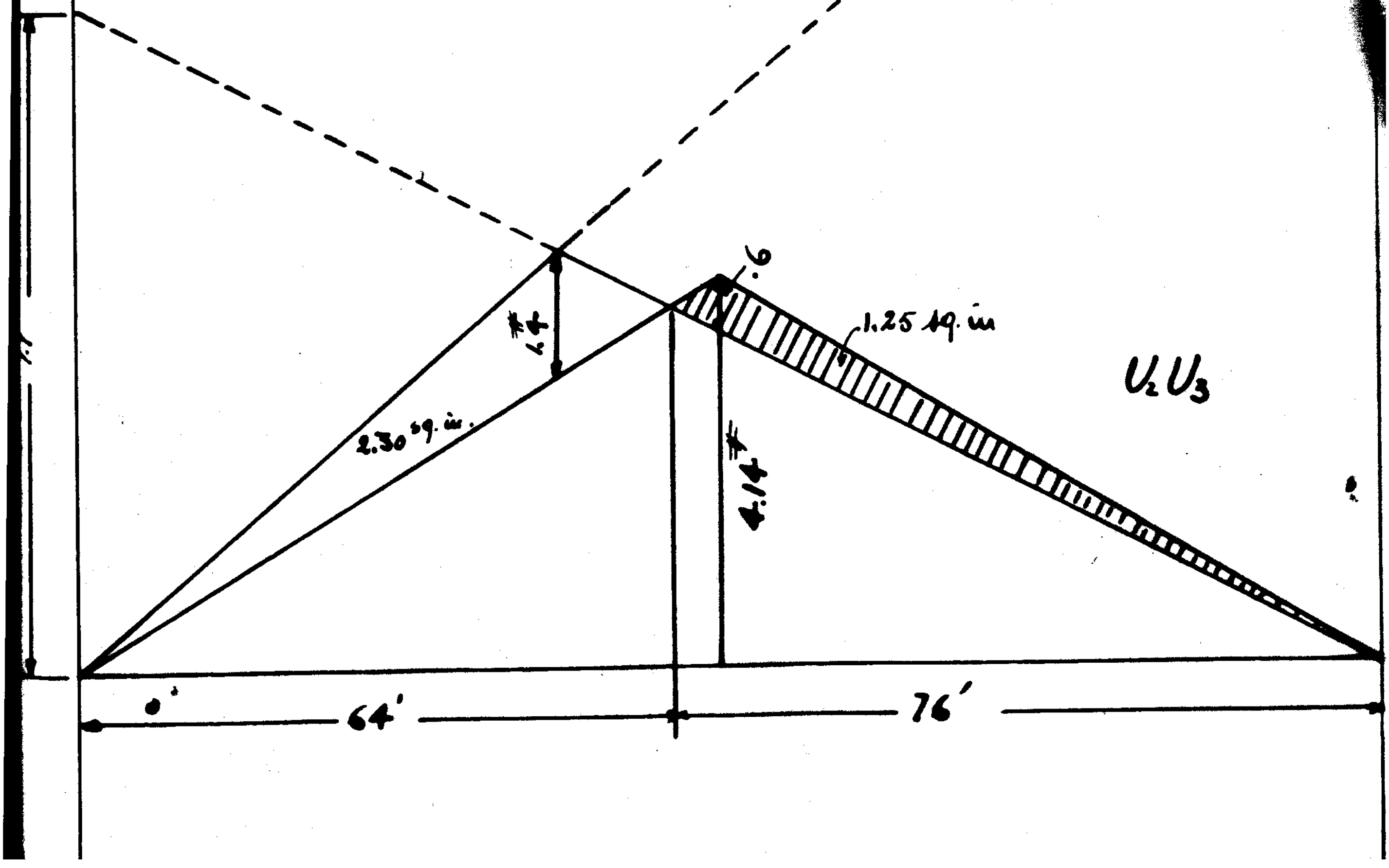
.70
1.08 sq. in
.29 sq. in
67
40'
100'



.52
.65 sq. in
.82 sq. in
51'
89'



U1U2
1.16#
.64#
1.32 sq. in.
1.52 sq. in.
2.05#
57.8'
82.2'



U2U3
1.4#
.6
1.25 sq. in
2.30 sq. in.
4.14#
64'
76'

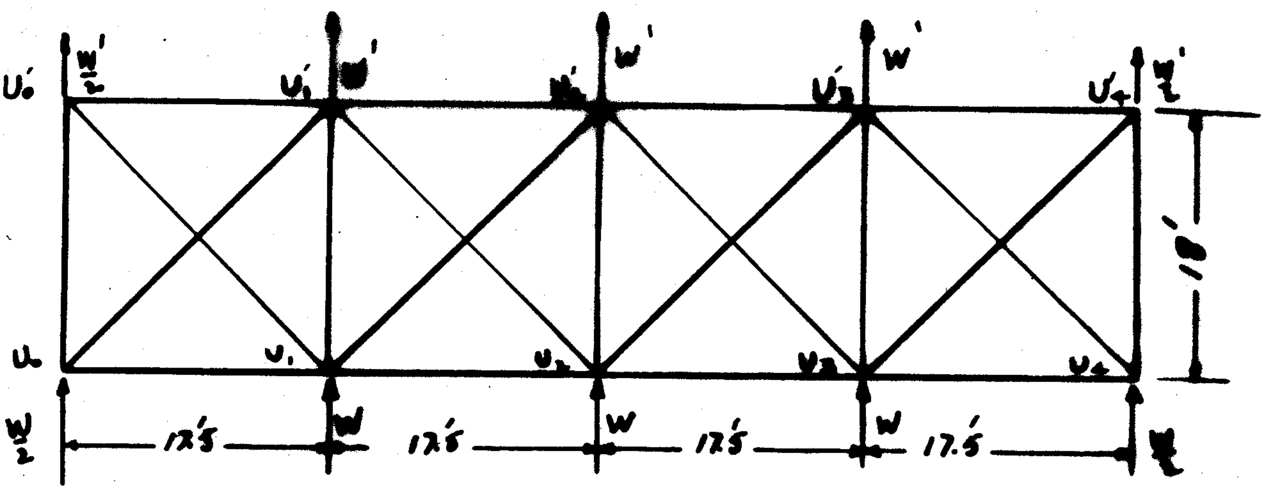
Stresses Due To Lateral Loads

Consider the three-hinged spandrel-braced arch in fig. D under the action of the wind loading shown. The upper chord is under the action of wind load of 500 lbs. per linear foot applied one-half on each side. When the hinge is placed in the plane of the lower chord at the crown, the upper lateral system cannot be made continuous from end to end of the structure, but must be interrupted at the crown to permit freedom of the hinge movement.

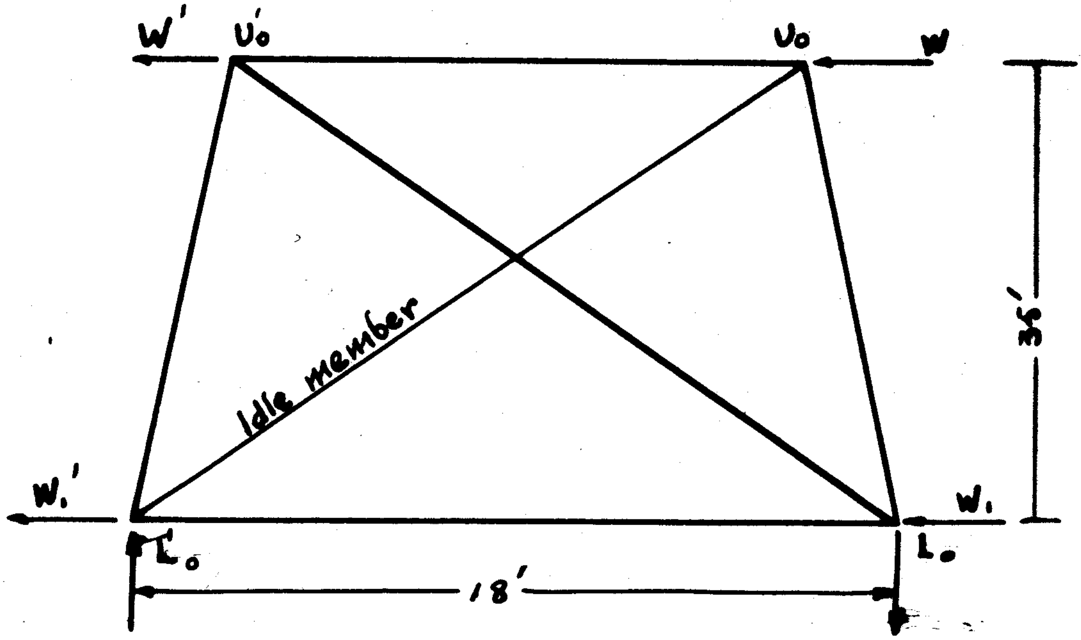
The upper lateral system therefore, can only transmit wind forces to the abutment hinge by virtue of its stiffness as a cantilever. By far the greater portion of these forces, however, is transmitted vertically at each panel point to the lower lateral system by virtue of the cross frames or vertical sway bracing at such panel points. The upper lateral forces W, transmitted to the lower system through the panel point crossframes, exert an overturning moment at the rib.

The lower lateral system may next be designed, assuming it to carry a wind load of 150 lbs. per linear foot on each side. Before the analysis is made, it is ofcourse, neccessary to develop the lower lateral system into a plane, as shown in Fig. D, after which it may be treated as a simple truss span whose panel length are the actual developed lengths of the inclined chord members,

The main truss may next be analyzed for the vertical wind components due to the overturning action as shown in diagram Number 4.



UPPER LATERAL SYSTEM



CROSS FRAME

<u>Computations For Wind Stresses</u>

On upper chord use 500 lbs./ linear foot applied ½ from each side.

On lower chord use 150 lbs./linear foot on each side.

$w' + W = 500 \times 17.5 = 8{,}750$ lbs.

$$\frac{17.5}{18} = .973$$

$$\frac{25.1}{18} = 1.395$$

$S\ (U_3 - U'_4) = 1.395 \times 4{,}375^{\#} = +\ 5{,}930$ lbs. Wind

$S\ (U_2 - U'_3) = 1.395 \times 13{,}125^{\#} = +\ 18{,}300$ lbs. Wind

$S\ (U1 - U'_2) = 1.395 \times 21{,}875^{\#} = +\ 30{,}400$ lbs. Wind

$S\ (U_o - U'_1) = 1.395 \times 30{,}625^{\#} = +\ 42{,}600$ lbs. Wind

$S\ (U'_3 - U'_4) = -\ 5{,}930$ lbs.

$S\ (U'_2 - U'_3) = -\ 23{,}730$ lbs.

$S\ (U'_1 - U'_2) = -\ 53{,}330$ lbs.

$S\ (U_o' - U'_1) = -\ 93{,}830$ lbs.

$S_{L_o} - U'_o = -30{,}624^{\#} \times 39.4/16 = \underline{75{,}400 \text{ lbs.}}$

$SU'_oL'_o = +\ 30{,}624 \times 35/16 = \underline{67{,}000 \text{ lbs.}}$

Bottom Bracing

$W_1 = W'_1 = 150\ (10.9 + 9.8) = 3{,}100$ lbs.

$W_2 = W'_2 = 150\ (9.8 + 9.2) = 2{,}810$ lbs.

$W_3 = W'_3 = 150\ (9.2 + 8.8) = 2{,}700$ lbs.

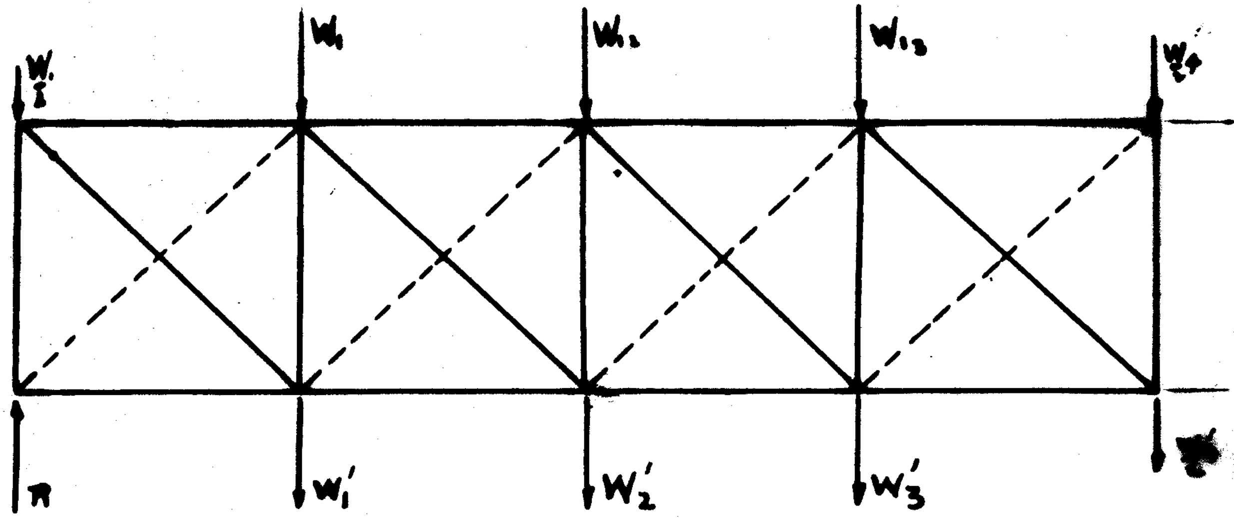
$W_4 = W'_4 = 150\ (17.6) = 2{,}640$ lbs.

$= 11{,}290$ lbs.

$R = 11{,}290 \times 2 = 22{,}580$ lbs.

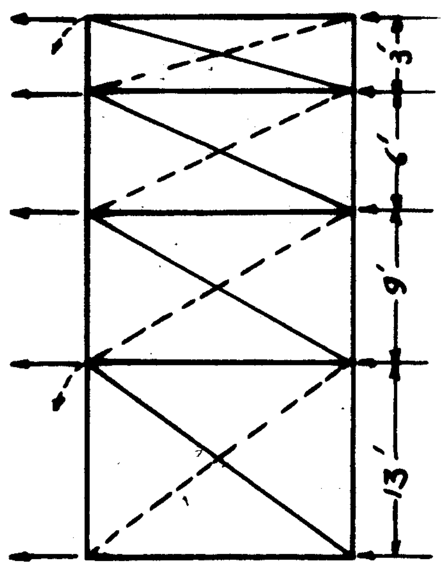
$S_{L'_oL_1} = 22{,}580 \times 28.2/18 = 35{,}300$ lbs.

$S_{L'_1L_2} = 16{,}380 \times 26.7/18 = 24{,}150$ lbs.



Lower Laterals _Plan

Dotted lines indicate idle members under ass
condition that diagonals carry no compression.
W. W, indicate total WINDWARD and leeward lateral



Lower Laterals _End Elev.

Arrows indicate direction WIND.
Dotted arrows indicate tendency of truss to rotate about each panel point.

$S_{L'_2L_3} = 10,680 \times 25.8/18 = 15,300$ lbs.

$S_{L'_3L_4} = 5,280 \times 25.2/18 = 7,400$ lbs.

$L'_oL_o = -22,580$ lbs.

$L'_1L_1 = -19,480$ lbs.

$L'_2L_2 = -13,530$ lbs.

$L'_3L_3 = -7980$ lbs.

$L'_4L_{4n} = -2,640$ lbs.

$S_{(L'_o - L'_1)} = 35,300 \times \frac{21.2}{28.2} = -26,600$ lbs.

$S(L'_1 - L'_2) = 24,150 \times \frac{19.7}{26.7} + 26,600 = -44,400$ lbs.

$S\ (L'_2 - L'_3) = 15,300 \times \frac{18.5}{25.8} + 44,400 = -55,400$ lbs.

$S\ (L'_3 - L'_4) = 7400 \times \frac{17.6}{25.2} + 55,400 = -60,560$ lbs.

$L_oL_1 =$ 0

$L_1L_2 = +26,600$ lbs.

$L_2L_3 = +44,400$ lbs.

$L_3L_4 = +55,400$ lbs.

Overturning Loads

Winds @ upper panel points

$P_1 = 8,750 \times 22/18 = 10,700$ lbs.

$P_2 = 8,750 \times 13/18 = 6,420$ lbs.

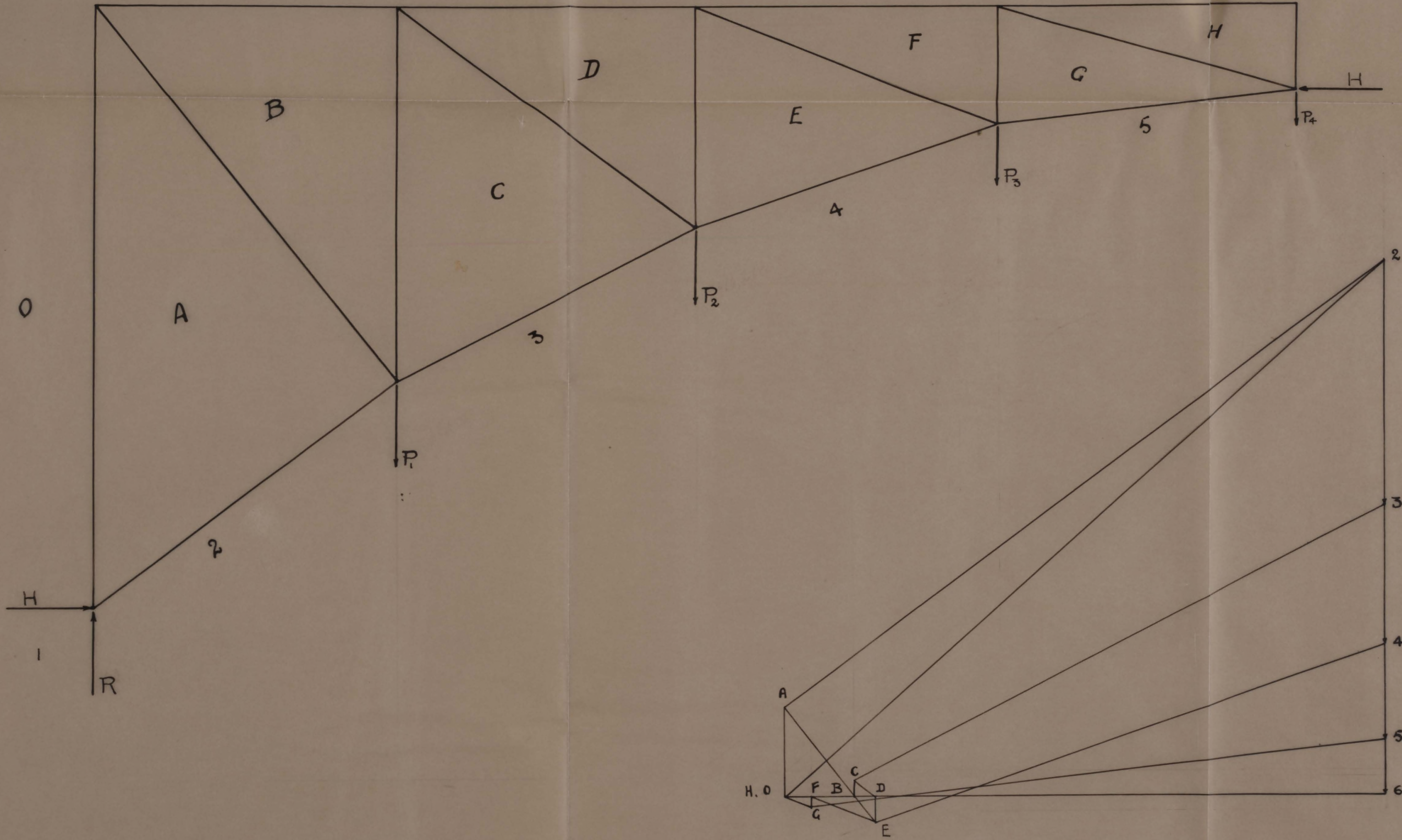
$P_3 = 8,750 \times 7/18 = 6,140$ lbs.

$P_4 = 8,750 \times 5/18 = 4,380$ lbs.

Overturning Loads (Stress in Diagonals)

$P_4 = \frac{5,280}{18} \times 2 = 587$ lbs.

$P_3 = 10,680 \times 6/18 = 3,560$ lbs.



NOTE:

$R = 62{,}877^{\#}$

$H = 70{,}200^{\#}$

$P_1 = 28{,}650^{\#}$

$P_2 = 16{,}260^{\#}$

$P_3 = 11{,}350^{\#}$

$P_4 = 6{,}617^{\#}$

GA. SCHOOL OF TECHNOLOGY
STRESS DIAG. FOR
VERTICAL WIND LOADS
ON ARCH RING

YEAR 1939 Y. PAVLIDIS

Scale 1" = 5' for TRUSS & 1" = 10,000#

Drawing Nº 4

$P_2 = 16{,}380 \times 9/18 = 8{,}190$ lbs.

$P_1 = 22{,}580 \times 13/18 = 16{,}300$ lbs.

Total Vertical Wind in Loads

$P_1 = 16{,}300 + 10{,}700 + 1{,}650 = 26{,}850$ lbs.

$P_2 = 8{,}190 + 6{,}420 + 1{,}650 = 16{,}250$ lbs.

$P_3 = 3{,}500 + 6{,}140 + 1{,}650 = 11{,}350$ lbs.

$P_4 = 587 + 4{,}380 + 1{,}650 = 6{,}617$ lbs.

Vertical Force Due to Load 6 ft. above Floor

Assume 8.5 ft. down to C.L. Top Chord

$8.5 \times 200 \times 77.5/18 = 1{,}650$ lbs. at each panel point.

Member	D.L.	L.L. using 480# per ft of truss + 13500# conc.		Impact percentage		Impact Stresses		Wind Load	Maximum Stresses Comp.	Members length in ft.	Maximum Stresses Ten.
		Ten.	Comp.	Ten.	Comp.	Ten.	Comp.				
L_0L_1	-189,000		127,371		20		25,474	-87,600	429,445	21.8	
L_1L_2	-172.000	4,110	119,500	33	21	500	25,100	-70,000	386,600	19.7	4,610
L_2L_3	-159.000	14,810	113,800	30	22.4	4,450	25,500	-63,200	361,500	18.5	19,260
L_3L_4	-152.200	19,310	144,400	27.6	24	5,350	34,600	-67,800	399,000	17.6	24,660
U_0U_1		20,130	19,100	23.4	28.6	4,700	5,470	-8,100	32,670	17.5	24,830
U_1U_2		34,040	44,900	24	27.5	8,200	12,400	-10,700	68,000	"	42,240
U_2U_3		32,100	63,300	25	26.5	8,010	16,800	-3,300	83,400	"	40,110
U_3U_4										"	
U_0L_1		35,600	26,750	28.6	23.4	10,200	6,250	13,200	33,000	28.1	59,000
U_1L_2		34,700	28,900	27	24.4	9,350	7,050	3,200	35,950	21.8	47,250
U_2L_3		62,000	6,750	30	22	18,700	1,490	-8,100	16,340	18.8	80,700
U_3L_4		114,316	55,300	25	28	28,600	15,500	-3,500	74,300	18.2	142,916
U_0L_0	-21.510	16,770	3,768	24	29	4,030	995	-10,800	37,073	35	20,800
U_1L_1	-32.200	11,830	43,600	25	26.2	2,960	11,400	-2,000	89,200	22	14,790
U_2L_2	-32.200	7,344	45,600	26	25.4	12,060	11,600	+3,000	89,400	13	22,404
U_3L_3	-32.000	9,430	30,186	30	22.3	2,830	6,750	+1,400	68,936	7	13,660
U_4L_4	-32,000								32,000	5	

TABLE OF STRESSES - MAIN MEMBERS OF ARCH

NOTE: STRESSES SHOWN IN POUNDS.

Table I

Members	Design Stress	Section Used	Properties of Sections				Total Unit Stress	Allowable Unit Stress	[illegible] ft.	Weight of [illegible]
			A	I	r_{x-x}	r_{y-y}				
L_0L_1	-429,445	A	30.36	687	12.80	4.67	14,200	15,000	[illegible]	1130
L_1L_2	-386,600	"	"	"	"	"	12,650	"	[illegible]	[illegible]
L_2L_3	-361,500	"	"	"	"	"	11,900	"	18.5	960
L_3L_4	-399,000	"	"	"	"	"	13,200	"	17.8	930
U_0U_1	-125,100	B	12.06	256.6	5.55	8.6	10,400	15,000	17.5	362
U_1U_2	-102,000	"	"	"	"	"	8,500	"	"	362
U_2U_3	-125,000	"	"	"	"	"	10,400	"	"	362
U_3U_4	-49,010	C	6.72	64.6	6.2	6.6	6,800	15,000	"	201
U_0L_1	+83,500	E	8.04	-	-	-	18,000	18,000	26	360
U_1L_2	+78,500	E	8.04	-	-	-	18,000	18,000	19.5	270
U_2L_3	+124,000	D	17.60	-	-	-	18,000	18,000	14.7	[illegible]
U_3L_4	+214,316	D	17.60	-	-	-	18,000	18,000	12	360
U_0L_0	-133,800	F	14.61	242	16.8	3.76	9,150	10,610	[illegible]	620
U_1L_1	-133,800	"	"	242	"	"	9,150	"	[illegible]	410
U_2L_2	-133,800	"	"	"	"	"	"	"	12.5	250
U_3L_3	-102,936	G	11.71	273	23.4	[illegible]	8,800	15,000	5.[illegible]	90
U_4L_4	-102,936	"	"	"	"	"	"	"	3.8	60

Section A: 2 [S] $18 \times 4 \times 51.9^{\#}$ E: 2 [S] $8 \times 2\frac{1}{4} \times 13.75^{\#}$

" B: 2 [S] $12 \times 3 \times 20.7^{\#}$ F: 4 Ls $3\frac{1}{2} \times \frac{3}{8} \times 4 \times 9.2^{\#}$

" C: 2 [S] $8 \times 2\frac{1}{4} \times 11.5^{\#}$ PL $10.5 \times \frac{3}{8}$

" D: 2 [S] $10 \times 2\frac{5}{8} \times 30^{\#}$ G: 4 Ls 3 × [illegible] × [illegible] × $6.6^{\#}$

DESIGN: MAIN MEMBERS OF ARCH PL $10.5 \times \frac{3}{8}$

Table II

Member	Stress for Design	Pro. of Sec. Gross A.	[illegible]	r	l/r	Section Used	Total length	Weight of Memb.	
U_0U_1'	+42,600	[illegible]	[illegible]	-	-	2 Ls 4 × 3 × 3/8	25.1'	430	
U_1U_2'	+30,400	3.8[illegible]	2.[illegible]	-	-	2 Ls 3 × 2½ × 3/8	"	334	
U_2U_3'	+18,300	3.10	1.90	-	-	2 Ls 2½ × 2 × 3/8	"	268	
U_3U_4'	+6,100	"	"	-	-	"	"	268	
$L_0'L_1$	[illegible]580	4.96	3.7[illegible]	-	-	2 Ls 4 × 3 × 3/8	28.2'	480	
$L'L_2$	+24,150	3.84	2.6[illegible]	-	-	2 Ls 3 × 2½ × 3/8	26.7'	380	
$L_2'L_3$	15,300	3.10	1.90	-	-	2 Ls 2½ × 2 × 3/8	25.8'	275	
$L_3'L_4$	7,400	"	"	-	-	"	25.2'	275	
$L_0'L_0$	-22,580	5.24	3.78	1.26	170	4 Ls 2½ × 2" × 5/16"	18'	390	
$L_1'L_1$	-14,440	"	"	"	"	"	"	390	
$L_2'L_2$	-13,530	"	"	"	"	"	"	390	
$L_3'L_3$	-7,980	"	"	"	"	"	"	390	
$L_4'L_4$	-2,640	"	"	"	"	"	"	390	
$U_0'L_0$	-75,400	16	13	2.5	189	2 Ls 8 × 8 × ½	39.4'	2080	

TOP AND BOTTOM LATERAL SYSTEM

Table III

MEMBER'S DESIGN OF THE STEEL ARCH

After finding the neccessary stresses for the arch action, the question of designing the members of the arch arises. Tables of stresses for various members have been made up and presented together in tables I, II and III.

In selecting the members care was taken to fix the dimensions in such a way that good joints could be made between the chord members and the web system. Sample computations have been reproduced below to show the manner in which the tables were built.

Design of L_0L_1 (Example)

Total Design Stress = 429,445 lbs. compression

Length of the Member = 21.8 ft.

Assume f_s = 15,000 lbs./sq.in.

Gross Area of Steel Required

429,445/15,000 = 28.7 sq.in.

Use Two 18" x 4" x 51.9 lbs. Channels (A_g = 30.36 sq.in.).

Distance Back to Back = 11 in.

Inertias on the two Axis

$I_{xx} = 622.1 \times 2 = 1244.2$ in.^{4} (Tables in code book of A.S. C.E.)

$I_{yy} = 17.1(2) + (4.63)^2(30.36) = 687$ in.^{4}

Radius of Giration

$r = \sqrt{687/30.36} = 4.67$ in.

$l/r = \dfrac{21.8 \times 12}{4.67} = 55.8$ (less than 60. f_s = 15,000 is O.K.)

All the members in compression are computed in the same manner as this example.

Design of Member U_3L_4 (Example)

Design Stress = 214,316 lbs. tension

f_t = Assume 18,000 lbs./sq./in.

Net Area of Member

214,316/ 18,000 = 11.9 sq.in.

7/8 Rivets are used throughout the whole design.

Use Two 10" x 2 5/8" x 30 lbs. Channels. (A = 17.60 sq.in.).

Area Required

11.9 + 6 x 1' x .673 = 15.95 sq. in. (O. K.)

The rest of the tension members are computed in like manner.

****

PART III

COST ESTIMATE AND ECONOMIC COMPARISON OF THE TWO BRIDGES

COST ESTIMATES OF THE TWO BRIDGES

THE REINFORCED-CONCRETE ARCH BRIDGE

Railing :	28.50	cubic yards of concrete.
Slab :	58.70	" " " " .
Beams :	23.40	" " " " .
Girders :	93.50	" " " " .
Columns :	3.50	" " " " .
Arch Rib :	111 .00	" " " " .

Total Cubic Yards of Concrete = 318.60

Paving = 1.4 of Bituminous Material

Pounds of Reinforced Steel = 95,000 lbs.

COST

318.60 cu.yds. of Concrete - Class A	@ $35	=$11,151
95,000 lbs. of Reinforced Steel	@ $ .06	= 5 ,700
1.4 cu.yds. of Paving		= 500
Sub Total		=17,351
Engineering and Sundry Expenses 10%		= 1,735
TOTAL COST		= $19,086.00

Seems low

THE STEEL ARCH BRIDGE

Main Members of Arch	33,588 lbs.
Top and Bottom Laterals	18,520 lbs.
Bracing Other than Lateral	27,140 lbs.
Sub Total	79,248 lbs.
40% for Details	31,700 lbs.
Railing and Stringers	27,140 lbs.
Shoes and Pins	15,000 lbs.
Total Steel	167,182 lbs.
Cubic Yards of Slab	55

COST

167,182 lbs. of Structural Steel	@ $.07	$11,702.80
55 cu.yds. of Concrete	$.35	1,925.00
Sub Total		13,627.80
Engineering and Sundry Expenses 10%		1,367.70
TOTAL COST OF STEEL ARCH BRIDGE		$14,990.50

ECONOMIC COMPARISON

After the preliminary estimates of quantities and first costs have been determined for the competing types of bridges, the next step is to find out which type is the more economical to build. A lowest first cost does not neccessarily mean the most economical. The service that the bridge ought to give in a certain length of time, is an important factor entering in the study of selecting the bridge.

In addition to the first cost of the bridge there are other items entering into the economic equation, such as maintanance, interest on first cost, operating cost, insurance and depreciation. Thus the true cost of the bridge is represented by the total average annual cost over a period of years equal to the length of life of the structure. The cost of maintanace should be determined from actual cost records for similar structures. The maintanance costs include items that are common to all types of bridges.

Maintanance cost for steel bridges include repainting all steel parts, repairing accessory joints and other small items. Maintanance costs common to concrete bridges are mostly concentrated in the repairing of damage done due to wheel guards and handrails. The total cost of maintenance in concrete structures is somewhat lower than any other type. However, if concrete construction is not carefully supervised, deteriortion may soon set in and increase the maintenance costs a great deal.

Interest on the first cost of a structure represents an annual expense, which probably amounts to more than the cost for maintenance unless the structure is built of timber.

Operating costs for bridges may be divided into two parts: those resulting in an expense to the state and those causing an expense to the public who use the bridge or structure. Both classes should be considered in economic analysis.

Insurance costs are only required in case of timber construction on account of fire and other hazards.

Annual depreciation is another expense which the state must meet when a bridge is built. This cost depends upon the economic length of life of the structure, the first cost and probably upon a rate of compound interest carried upon a yearly deposit. There are two methods of arriving at the annual depreciation of a structures: In the annuity method, the amount of money is computed which is neccessary to deposit at the end of each year at a given rate of compound interest, to amount to suffise to rebuild the structure at the end of its economic life. The second method of finding the depreciation is to neglect any amount of compound interest which may be earned on yearly deposits and divide the first cost on the number of years of economic life of the structure.

The formula which gives the total average annual cost of bridges is as follows:

$$\text{Annual Cost} = C\ (D/C + M/C + I + O/C + i/C) - (C - W)/N$$

where N = Economical length of life of structure.

D = Average annual depredation.

M = Average annual cost of maintenance.

I = Average interest on first cost expressed in percentage.

O = Average annual operating cost.

i = Average insurance cost.

C = Total cost of structure.

W = C - salvage value.

A concrete bridge does not have a salvage value except in cases where the old bridge may be broken up and used as rip rap to protect a new structure; even then, its value is so small that no allowance is made for it in an economic comparison. A steel bridge however, may be torn down and parts of it may be used again in smaller structures. The remainder of t thh steel may be sold as scrap. The salvage value of the steel truss, depends upon its location.

Aestetics is an item which is always considered, either conscciously or unconsciously, in the design of any bridge. The amount of the aesthetic value depends upon the distance from a center of population, the land-scape in the vicinity of the bridge, the architectural treatment and the chance which passing people will have in getting an unabstracted view of the structure.

In the selection of the bridge, over Sakarya river, considering all the items mensioned above, it is a great advantage to the builder in selecting the steel bridge. The location of the bridge is not in any important center and besides the profits made over the concrete bridge by using the former is 4,000 dollars.

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